THE UNIVERSITY OF ALBERTA

# VOLUME AND STRENGTH CHANGES IN SOILS DUE TO FROST ACTION

A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES

IN PARTIAL FULFILMENT OF THE REQUIREMENTS

FOR THE DEGREE OF MASTER OF SCIENCE

DEPARTMENT OF CIVIL ENGINEERING

by

WALTER JOHN ROSS

EDMONTON, ALBERTA

APRIL, 1960

## ABSTRACT

Prior to the development of the principles of compaction by Proctor in 1933, soils were placed in fills with little regard for water content and density. With the introduction of compaction, the question arose as to whether or not a compacted soil maintained its compacted density.

It was the object of this investigation to study the behaviour of four typical Alberta subgrade soils compacted to standard and modified Proctor densities and then being subjected to nine freeze-thaw and capillary saturation cycles. The behaviour of a base course material was investigated up to six cycles. In each case the samples were frozen and thawed in a system in which no external water was available. Between thawing and freezing the samples were permitted to become saturated by capillarity.

From the results of the tests carried out it was found that all samples decreased in strength, the greatest loss occurring during the first two cycles. It appears that changes in moisture content are much more critical in the determination of strength changes than are the densities. In each case, the soils compacted to the higher density maintained a greater strength than those compacted to a lower density. However, the results for an inorganic clay of low plasticity indicated that the strength of the modified Proctor density samples would eventually be the same as the standard Proctor density samples. Thus, for the soils which were tested, other than

this type, it is recommended that they be compacted to the maximum possible density. There appears to be little advantage in compacting the inorganic clay of low plasticity which was tested to a density higher than standard Proctor density.

## ACKNOWLEDGEMENTS

The author wishes to express his appreciation to:

Professor S.R. Sinclair and Mr. B.P. Shields for their helpful criticism and guidance throughout.

Alberta Research Council under whose sponsorship this investigation was carried out.

Department of Highways, Province of Alberta, for the provision of the soils.

Canadian Good Roads Association, to whom the author is indebted for his graduate studies.

TABLE OF CONTENTS

## LIST OF PLATES

## Chapter 1 - INTRODUCTION

Prior to the introduction of the principles of soil compaction, which indicate the water content at which a soil should be compacted for a given compaction effort in order to obtain the maximum possible density, soils were placed in fills without regard to the water content and the compaction effort. Although the principles are not fully understood, they govern the placement of the majority of earth fills for the following reasons: to minimize settlement as well as differential movement; to increase the shear strength; and to decrease the permeability. On the other hand, the benefits to be gained from compaction have been questioned on the basis that compacted soils may revert back to their uncompacted density when they are subjected to alternating freezing and thawing in an open system. (i.e., a system in which external water is available during the freezing process). However, it should be realized that this is an extreme condition, and that the general condition lies somewhere between an open system and a closed system. (i.e., a system in which no external water is available during the freezing process).

A decrease in density of a compacted soil may be attricuted to an increase in colume due to frost action or swelling. In order to study these conditions, a closed system was employed in which the samples were frozen from all directions at -30°C to minimize the formation of ice lenses. Between each thaw and freeze cycle, the samples were

permitted to become staurated by capillarity. It is believed that this testing procedure would represent a more severe condition then would exist in the field in which the formation of ice lenses is at a minimum and in which an external source of water is available during the period when an external source of water is available during the period when the soil is unfrozen. The soils which were investigated covered a typical range of materials from a granular material to an inorganic clay of high plasticity, from the Province of Alberta (Plate 2).

Frost Action

With the organization of research groups, such as the Highway Research Board in the early 1920's, and the growing demand to minimize the damage due to frost action, investigators undertook to determine the physical processes associated with soil freezing. It was realized that the damage caused by frost action could take place in two ways. The actual heave, if non-uniform, may produce detrimental effects where differential movement is a critical factor. The secondary effect of softening which accompanies thawing of ice layers may bring about a reduction in load carrying capacity.

Early investigators attributed the damage due to frost to the formation of ice lenses. This may be summarized according to Beskow[2] as follows: " .... in ordinary cases freezing of soil without capillary connection with free ground water is of not significance in road and railroad problems". However, recent investigations have indicated that the bearing value of a soil may be reduced to the order of 50%

by a single freeze-thaw cycle, without any appreciable change in moisture content or density of the material[12,13]. It is believed that the loss may be due to the distribution of the water in the soil.

On freezing, soils may appear as a homogeneous or as a stratified frozen soil. A homogeneous frozen soil is one in which water is frozen within the natural voids and there is no visible accumulation of ice lenses. A stratified frozen soil contains visible ice lenses which occupy spaces greater in size than the original voids.

In nature, soils usually appear as a stratified frozen soil in either an open or a closed system. However, if freezing occurs at such a rate that the movement of water is prohibited a homogeneous frozen soil will prevail.

According to Taber[1] and Beskow[2], soils in nature seldom behave as absolutely closed systems on freezing. Taber[1] mentioned that a closed system may occur when the water table is parallel to the surface and the soil water begins to freeze simultaneously over a large area. The resistance to the movement of ground water due to the air being excluded from the soil may be so great as to give the effects of a closed system. Soils, such as bentonites and gumbos, may also behave like closed systems on freezing due to their imperviousness. In fact, there is no sharp demarcation in the conditions that cause soils to freeze as an open rather than a closed system.

Even during the freezing of a soil, the resistance to the movement of the ground water may increase so that an open system tends to grade into a closed system.

Frost Action in Open Systems

Prior to the development of Taber's[1] theory of ice lens formation in the early 1920's, it was believed that the heave in soils was due solely to the expansion of the water in the soil on freezing. Taber showed that the total heave could be contributed to the formation of ice lenses. During the formation of ice lenses water is supplied to the crystals through small capillary passages, the uplift being due to the cohesive forces in the water. He held that, "Conditions existing during the growth of an ice layer in clay are particularly favorable for the uplift of water by molecular cohesion".

From the results of tests carried out by Benkelman and Olmstead[3] in 1931, based on a fluctuating frost line, it was tentatively concluded that excessive heaving may occur in soil of almost any grading or texture provided an adequate supply of water is available. According to this theory all that is required to produce ice lens formation is a saturated soil and a fluctuating frost line. However, this theory is not generally accepted although it helps to explain the discrepancy between Taber's theory and the behaviour of soils found in the State of Michigan.

The explanation of frost heaving presented by Beskow[2] in 1935 was, in general, in agreement with the theory of ice lens formation advanced by Taber[1] . He believed that in freezing a soil in which ice is forming ".... the water next t8 the ice strata changes from a liquid to a solid state and has the same effect as a change into vapor by evaporation".

## Frost Action in Closed Systems

A closed system infers that the water for ice lens formation must come from that held within the soil and the soil does not have available an external source of water. The migration of water towards the freezing zone occurs without an overall increase in water content. Beskow[2] found that in a closed system in which a silt sample was saturated and frozen in one direction, the resulting heave was usually small. However, it is significant that freezing produced a marked increase in the water content in the freezing zone, causing a reduction in strength on thawing. From similar experiments on a partly saturated sand, he found that the heave was several times that which may be attributed to the volume change of the contained water on freezing, and that the expansion was affected by the surface loading. According to Taber[1] , when freezing occurs in a closed system, pressures are developed which produce heave due to the expansion of the water on freezing, and the amount of heave varies with the amount of water available.

## Effect of Density On Frost Action

The capillarity and permeability of a soil are influenced by the total porosity and by the size of the soil pores. Beskow[2] found that the density affects the capillarity of soils, and thus heaving, since frost action depends on the rate at which water can be drawn to the ice lenses. In 1940, Winn and Rutledge[4] carried out experiments on a saturated sandy clay in an open system to evaluate the effect of density on frost action. These studies indicated a critical density below which frost action is directly proportional to density, and above which frost action is inversely proportional to density. It was concluded that, "increasing the density above the critical density increases the period of inactivity before heaving starts and decreases the rate of heaving and the total heave in a manner similar to the addition of admixtures".

From an investigation by the Arctic Construction and Frost Effects Laboratory[26] on soil types ranging from clay to sandy gravels, the rate of heave was found to be responsive to changes in density as well as with soil type. However, it was concluded that ...."it is not necessarily obvious whether an increase in degree of compaction in a given soil should result in an increase, or in a decrease, in the rate of frost heave in the absence of experimental test results."

Review of Tests on Densified Soils

Since the development of soil compaction, several attempts have been made to determine the effect of freezing and thawing on a densified soil. In 1940 Lang[6] carried out tests on four different types of soil compacted at different water contents in which the samples were carried through a series of freeze-thaw cycles. Each cycle consisted of saturating by capillarity the samples for a period of five days and then freezing them in an open system. After three cycles there did not appear to be any further appreciable change in the dry density, and that under such test conditions a densified soil may revert back to its uncompacted density.

In 1942, Campen and Smith[7] carried out tests on a wide range of soil types in order to assess the durability of densified soils. Individual samples were subjected to drying and freezing, from all directions, and capillarity tests in which the samples were permitted to take up water from the bottom of the sample for a period of seven days. Since the tests were carried out for only one cycle, the results are of limited value. One point of interest which occurred was that on freezing the samples in a closed system, some soil mixtures contracted while others expanded at optimum water content. It was found that the shrinkage was much greater than that calculated on the basis of the contraction co-efficients of quartz, water, air and ice. This was explained on the basis that the films of water on the soil particles may be in the form of free or solid water. Since the shrinkage as determined

from the co-efficient of contraction was much less than the measured values, it was concluded that the films possessed very high co-efficients of contraction and that the degree of contraction varied with the thickness of the film.

An extensive coverage of the drying and wetting process was carried out by Porter[8] in the early 1940's. He found that the permanency of the original density of a clay soil depended mainly on the water content of the soil at the time additional water came in contact with it. When the clay soil specimens were carried through several cycles of wetting to saturation and drying to constant weight at 100°F, each specimen returned practically to its first dry density and to its first wet density in every cycle. The total fluctuations in the densities between the dry and the saturated conditions were greatest in the most dense specimens and smallest in the least dense specimens. According to Porter the more dense a clay soil was compacted, the smaller was the percentage of water it absorbed to become saturated and that the specimens that absorbed the least percentages of water had the greatest strength. Although the drying effect may be serious in a dry climate it would be practically negligible in a climate in which frost action and high precipitation are the predominant destructive factors.

Frost Susceptible Soils

The extent of frost heave occurring for a given frost

penetration in a given soil is frequently regarded as the criterion of danger from frost. However, this criterion may be considered to be incomplete in that it does not account for the weakening of the subgrade soils, in which little or no heave occurs, during the thaw period. With the exception of grain size distribution only limited emphasis has been placed on the relationship between soil classification and frost action. However, recent investigations have indicated that when a clay mineral is present in a soil in a minor amount, its effect may be different than when the percentage of the mineral is high enough so that its properties are predominant.[26]

With regard to the identification of frost susceptible soils based on grain size distribution, the Casagrande[10] frost criteria has been used extensively on the North American continent. This criterion considers soils to be frost susceptible if more than 3% by weight of the soil fraction is less than 0.02 mm for non-uniform soils or 10% is less than 0.002 mm for very uniform soils. In 1948 this criterion was modified, recognizing qualitavely six degrees of potential frost action, based on the Airfield Classification System.[15]

In recent years it has been confirmed by research that many cases of frost damage can be attributed largely to the very fine sand fraction and that only the portion of the soil less than 2 mm is involved in an assessment of frost danger.

Schaible[11], taking this into account has proposed a frost criterion based on the fraction of fine soil less than 2 mm. According to this proposal, the soil may be classified into a clay and silt series, or in a clay, silt and very fine sand series. By using this classification along with the percent finer than the 2 mm fraction, the soil may be placed in one of four potential degrees of frost susceptibility.

The soil is described as dangerous if more than 20 percent of the soil fraction is less than 2 mm for the clay and silt series, and if more than 40 percent is less than the 2 mm for the clay, silt and very fine sand series.

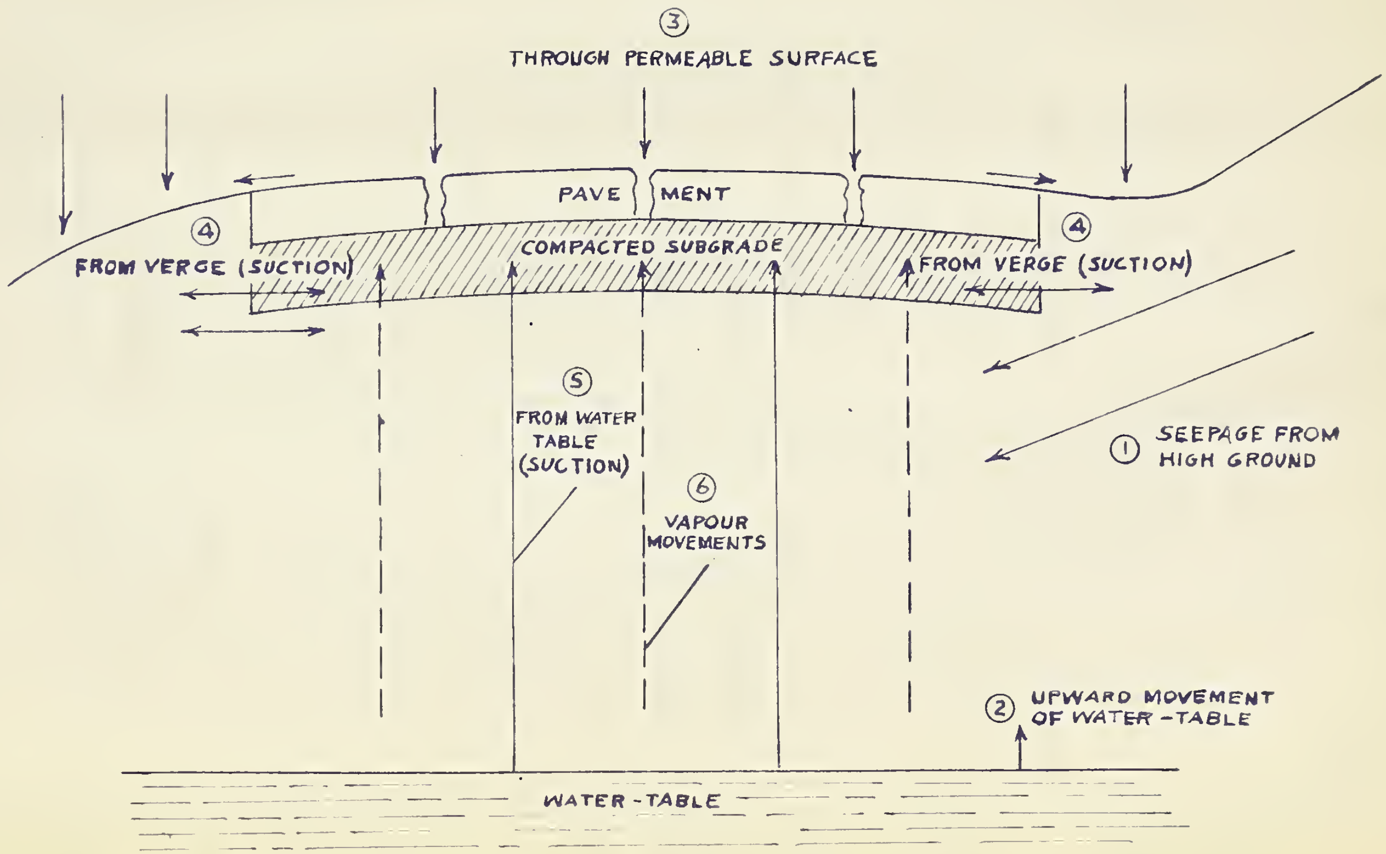
Moisture Sources

The regular variations in moisture and density in subgrade soils are due to the annual changes in climatic conditions: soil moisture increase in the fall; moisture migration in the subgrade during freezing; thawing and drying of the subgrade. As may be seen from Plate 1, the source of additional water depends on the conditions existing in the field.

From field studies of the movement of soil moisture on sand, silty clay, and heavy clays, Black, Croney and Jacobs[14] found that seasonal changes of pore pressure and water were large under grass cover, but only small when the grass cover was removed. This shows that the transpiration from vegetation is more important than surface evaporation in removing water

from the soil. Under impervious pavements only small changes of pore pressure and water were found. The ultimate pore pressure was found to depend largely on the position of the water table.

From an analysis of highways in Germany[11] in a region which is predominantly hilly and mountainous, and in which perched water tables are very common, it was concluded that the precipitation, as well as the water seeping in from the surface and the sides, had more effect on the moisture of the soil than the water sucked up from below by capillary action. In all instances in which there was more than four feet between the water table and the roadbed, no danger due to capillarity could be observed. The investigations in these areas also showed that the most severe frost damage could be attributed to water seeping in from the surface or from the side.




WAYS IN WHICH WATER CAN ENTER AND LEAVE ROAD SUBGRADES
(AFTER ROAD RESEARCH LABORATORY - 1952)
PLATE I

## Chapter II - UNSATURATED STRENGTH THEORY

Previous to the co-operative triaxial shear research program by the Waterways Experiment Station[16], no particular attention was paid to the strength of unsaturated soils. From the shear tests carried out on saturated and unsaturated samples it became obvious that the values of cohesion and angle of internal friction depended on the drainage permitted during the shear test. Taking this into account, Casagrande[16] stated a hypothesis for the effects of volume change and pore pressure on the behaviour of cohesive soils in shear. This hypothesis led to three specific types of triaxial tests: the drained test, in which all stresses in the soil sample are effective during shear; the consolidated undrained test, in which the soil sample is permitted to consolidate under an all-round pressure and then sheared so rapidly that no volume change can take place; and, the undrained test in which the all-round pressure is rapidly applied, and without allowing any consolidation, the sample is sheared to failure.

With respect to the findings on unsaturated cohesive soils[16] it was concluded that there was no simple relationship existing between the partially saturated and the saturated strengths. However, it was found that the soils in the partially saturated group could be identified by the following shear test characteristics:

1) In the undrained test the principle stress difference increases with lateral pressure.

2) The strength values obtained by the undrained and the consolidated-undrained test are essentially the same.

3) The relation between compressive strength and the water content at the end of the test varies with the minor principle stress. When the strengths are plotted against water content, the points fall on curves that are displaced for each different lateral pressure.

The unsaturated strength theory becomes of great importance when dealing with compacted soils. Although the theory is imperfectly understood, the following variables are known to be of outstanding importance: lateral support, dry density, water content, and the degree of saturation. From a study of compacted clays, Leonards[22] found that on failure the strength is a function of the void ratio. Whether or not this is the case, the difficulty of determining the void ratio at failure makes this an impractical approach.

It has been reported by Foster[9] that for a given water content, an increase in dry density results in an increase in strength as measured by the CBR (California Bearing Ratio) test, up to a certain density, after which an increase in dry density results in a decrease in CBR values. The strengths obtained from the unconfined compression test have also been found to follow the same trend.

A more rational approach to the strength of unsaturated soils has been developed by Bishop and Henkel[17] . This concept

takes into account the compressibility of both the water and the air surrounding the solid particles. By evaluating the pore pressure, the maximum shear resistance ($\tau_f$) may be determined, for all practical purposes, by the expression:

$\tau_f = C' + (\sigma - u) \tan \phi'$, in which,

$c'$ = apparent cohesion - in terms of effective stress

$\phi'$ = angle of shearing resistance - in terms of effective stress

$\sigma$ = total pressure normal to the plane considered

$u$ = pore pressure

In problems connected with the undrained shear strength fo soils, the excess pore pressure ($\Delta U$) which develops under the changes in the major principal stress ($\Delta\sigma_1$) and the minor principal stress ($\Delta\sigma_3$) may be expressed as:

$$\Delta U = B\left[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)\right]$$

The co-efficients A and B are referred to by Bishop and Henkel as the "pore-pressure co-efficients". The value of B depends on the compressibility of the water alone and for partially saturated soils is found to vary within the limits of zero and one. It has been found that the values of B range from about 0.1 to 0.5 for soils compacted to the maximum density at the optimum water content. The value of A depends largely on whether or not the soil is normally consolidated or over-consolidated, and on the proportion of the failure stress applied. If soils behaved according to the elastic theory, the co-efficient A would be 1/3 as determined

in the triaxial test. However, it varies with the stresses and strains and may be only quoted for a required point. Typical ranges of A for positive total stress increments are as follows:

Normally consolidated clays + 1/2 to + 1

Consolidated sandy clays + 1/4 to + 3/4

Lightly over-consolidated clays 0 to 1/2

Compacted clay-gravels -1/2 to + 1/4

Since the value of B varies throughout the stress range, for **partially** saturated soils, it may be more convenient to separate the product AB and to denote it by $\bar{A}$, and to express the excess pore pressure in the form:

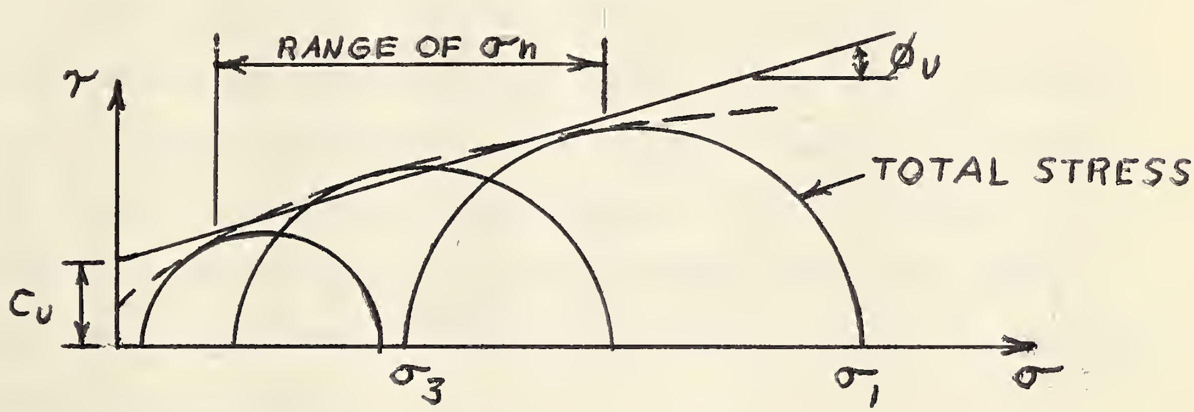
$$\Delta U = B \cdot \Delta\sigma_3 + \bar{A}(\Delta\sigma_1 - \Delta\sigma_3)$$

Undrained Test on Partly Saturated Cohesive Soils[17]

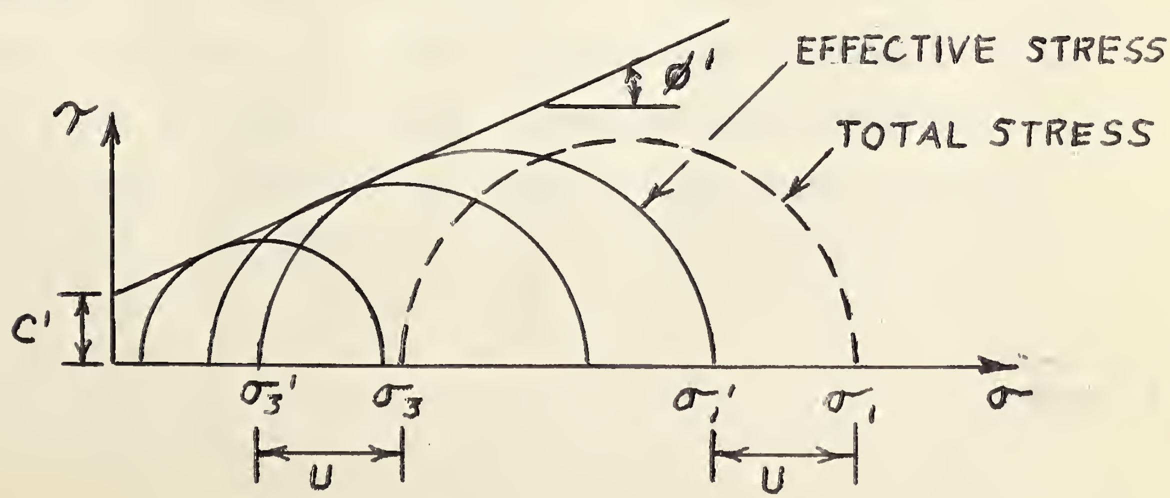
The most common application of the undrained test is in evaluating the strength of earth fills. When this test is carried out in the form of a triaxial test, the deviator stress at failure increases with the cell pressure. As the air in the voids is compressed and passes into solution, the increase in the deviator stress becomes progressively smaller, and ceases when the stresses become large enough to cause saturation. In this test, the pore pressure co-efficient B may be determined as the pore pressure recorded when the all-round pressure is applied. The co-efficient A is then

obtained from the pore pressure change during the application of the deviator stress.

Since the pore pressure changes are not uniform, unless saturation occurs, the failure envelope expressed in terms of total stress is non-linear, as shown in the following figure. Thus, the values of cohesion (Cu) and friction (∅u) in the undrained test can be quoted only for specific ranges of pressure.




If the pore pressure is measured during the test, the failure envelope can be expressed in terms of effective stress and is found to approximate very closely to a straight line over a wide range of stress, as shown in the following diagram.



In soils in which there is a tendency to dilate on shearing, values of C' and Ø' almost equal to their maximum values are found to be mobilized at a small fraction of the strain required to produce the maximum deviator stress. The increase in deviator stress after this point is almost entirely the consequence of the drop in pore pressure. A prolonged drop in pore pressure will result in values of C' and Ø' based on the maximum deviator stress being slightly less than their peak values. However, the practical significance of the difference of the stress circle to be used is usually negligible. In the case where C' is zero, the effective stress circle tangential to the limiting envelope has the maximum principal stress ratio $\sigma_1'/\sigma_3'$ . Thus, the circle can be identified directly from the tabulated results of the test.

<u>Drained Test on Partly Saturated Soils</u>[17]

In the standard test consolidation is permitted to take place under an equal all-round pressure, and the sample is then sheared by increasing the axial load at a sufficiently slow rate to prevent the build-up of any excess pore pressure. The minor principal stress $\sigma_3'$ at failure is thus equal to the consolidation pressure, and the major principal stress $\sigma_1'$ is the axial stress. Since the pore pressure is zero, the effective stresses are equal to the applied stresses. The strength envelope, in terms of effective stress, is

obtained directly from the stress circle at failure. The drained test may also provide information on the volume changes which accompany the application of the all-round pressure and the deviator stress, and on the stress-strain characteristics of the soil. If an increase in water occurs, the drained test may show a reduced value in apparent cohesion.

Unconfined Compression Test

The changes in strength of the subgrade soils in this investigation were measured by the unconfined compression test, which is the limiting case of the triaxial compression test in which the confining pressure is zero. Although it is carried out in the same manner as the undrained test, similar strength values can be expected only when there is no dissipation of the excess pore pressure. This condition may be considered to exist in the case of highly plastic clays. In clays of low plasticity, considerable dissipation of pore pressure may occur and the test conditions will be between the drained and the undrained test conditions.

If the angle of friction equals zero, the diameter of the stress circle corresponds to the maximum compressive stress. The cohesion is equal to half this stress. The unconfined compression test provides all the information required for a "$\phi = 0$ analysis" in a saturated clay, but is misleading in dilating or partly saturated soils in which the angle of shearing resistance is not equal to zero. For

soils far from saturation, an appreciable angle of shearing resistance would be exhibited, and the test does not give the full change in immediate strength for all the conditions of stress[18]. The unconfined compressive strength increases with the dry density and decreases with water content.

Multi-Stage Vacuum Triaxial Test

In order to evaluate the changes in strength of the granular material a multi-stage vacuum triaxial test was employed, due to the size of the samples. The fact that the values of C' and Ø' remain almost constant over a wide range of strain provides a means by which the failure envelope can be determined from one soil sample by using a multi-stage triaxial test[19]. The sample is failed, as in the triaxial compression test, under different confining pressures.

In the first part, or first stage, the vacuum triaxial test was run under zero confining pressure until the sample began to yield. Consecutive stages were then carried out under 0.2, 0.4, 0.6 and 0.9 atmospheres of pressure. After the last stage, the confining pressure was reduced to zero and the test was re-run in order to check on the amount of disturbance. If this stage showed agreement with the first stage it was evident that there has been no appreciable change of structure.

Since there was no control over the drainage, it was not known to what degree the test corresponds to the drained test. However, due to the type of test and the material which was tested, it is believed that the results would be approximately the same as those of a drained test.

## Chapter III - PROCEDURES

### Location and Classification[15] of Soils Tested

The subgrade soils which were tested in this program were chosen from areas throughout the Province of Alberta, with the intention that they would represent the general subgrade soil types located in these areas. Each of the materials was classified according to the ACS[15] (Airfield Classification System) with the exception that the soils with liquid limits between 30 and 50 were considered as intermediate plastic clays. The symbols which are used to designate the soil types are referred to throughout the report (Plate 3). The frost susceptibility of the soils is based on the Casagrande frost criteria.[15] The classifications are as follows:

1) Inorganic clay of low plasticity (CL soil) from the High River area; medium to high frost susceptibility.

2) Inorganic clay of medium plasticity (CI (1) soil) from the Castor area; medium to high frost susceptibility.

3) Inorganic clay of medium plasticity (CI (2) soil) from the Falher and Hythe area; medium to high frost susceptibility.

4) Inorganic clay of high plasticity (CH soil) from the Drayton Valley area; medium frost susceptibility.

A granular material from the Crawford Pit, near Stettler, Alberta was also tested. Due to the test procedure which was employed, it was necessary to use only that portion

of the material which was finer than 3/4 inch. This portion of the material was classified as a well graded gravel with a good clay binder. This material may be considered to be subject to medium frost action, according to the Casagrande frost criteria.[15]

Preparation of Soils

Preliminary testing consisted of performing plastic and liquid limit tests on each of the samples as a means of classifying them into their respective groups. Each of the samples consisted of approximately 500 pounds of soil. In order to obtain the greatest uniformity possible the following procedure was followed for each of the subgrade materials:

1. The soil was spread out over a large area, permitting it to become air-dried.
2. The air-dried soil was then placed in a concrete mixer which was rotated until it became relatively uniform in consistency.
3. On removal from the mixer, it was placed in an abrasion tester together with the metal balls which are used in the standard abrasion test.
4. When the larger clumps of material had been thoroughly broken up in the abrasion tester, the soil was removed and put through a Number 4 sieve.

5. The material passing the Number 4 sieve was then placed in a large container, and was again thoroughly mixed by hand.
6. The soil was then placed in bags and stored for future use.

In the case of the gravel, the same procedure was followed, with the following exceptions.

1) The abrasion tester was not used.

2) Only the material which was finer than the 3/4 inch screen was used for the test procedure on the gravel.

Standard Tests

The following tests were carried out on representative samples for each of the materials: Liquid limit, plastic limit, grain size analysis, specific gravity, standard and modified Proctors, and except for the gravel, the consolidation test. The results of these tests are summarized on Plate 2 with the exception of the consolidation test results.

Liquid and Plastic Limits Test

For each of the soils, a minimum of three liquid and plastic limit tests were carried out in order to determine the consistency of the soils as well as to serve as classification tests (Plates 2 and 3). It was found that the variation in the limits was less than one percent, except for the CH material. The procedure followed was that outlined by the ASTM (American Society for Testing Materials) for the determination of the liquid and plastic limits[21].

Specific Gravity Test

On each of the soils, two specific gravity determinations were carried out in accordance with the ASTM procedure[21]. The valves were found to be identical for each soil.

Standard and Modified Proctor Test

The moisture-density relations were performed in accordance with the ASTM procedure[21] with the following exceptions. A fresh sample of soil was used for each moisture content, and the time required for soaking was extended to 12 hours.

Consolidation Test

Consolidation tests were carried out for each of the samples, with the exception of the gravel. The consolidation specimens were obtained from samples which were compacted to the standard Proctor density. The procedure followed was that outlined in "Soil Testing for Engineers"[20] with the exception that the consolidation ring was of a semi-floating type. The consolidation curves are shown on Plates 12, 13, 14 and 15.

Grain Size Analysis

Three grain size analysis were carried out for each of the samples in order to check the uniformity of the samples as well as to determine the grain size distribution. (Plate 4). Within the range of experimental accuracy, there

appeared to be no variation in grain size distribution for a given type of soil. In order to determine the grain sizes, the sieve analysis was carried out for the granular soil, whereas a combined analysis was carried out for the subgrade soils. These were in accordance with the procedures outlined in "Soil Testing for Engineers"[20].

Preparation of Test Specimens for Cycling Procedure

Each soil was prepared to form two series of tests, one at the standard and the other at the modified Proctor density. Water was added to bring the moisture content of the soil as near as possible to its optimum for the given compaction effort, as determined from the moisture density results (Plate 2). The water was permitted to soak into the soil for not less than 12 hours. During this period the soil was kept in the moisture room to prevent evaporation.

The subgrade soils were compacted into six molds (6.16 inches in diameter by 5.90 inches in height) for each series of tests. To simulate the compaction effort used in the standard Proctor test, the soil was placed in the molds in four layers and was compacted with 57 drops per layer of the standard Proctor hammer. For the modified Proctor compaction effort, the soil was placed in six layers and compacted with 63 drops per layer of the modified Proctor hammer.

The granular material was compacted into six molds

(6.00 inches in diameter by 7.00 inches in height) for the standard Proctor density and in twelve molds of the same size for the modified Proctor densities. The standard Proctor compaction effort was obtained by placing the soil in five layers with 52 drops per layer of the standard Proctor hammer. The modified Proctor compaction effort was obtained by placing the soil in seven layers with 62 drops per layer of the modified Proctor hammer.

An attempt was made to minimize the side friction between the mold and the soil during the cycling process by coating the molds with two layers of Amber Parmo grease separated by cellophane, which was also used to separate the soil from the grease. The molds were clamped on a perforated base plate, covered with filter paper, and the initial weight of molds plus bases were determined to the nearest gram with the scales shown on Plate 5. The volume occupied by the grease and cellophane was neglected in the determination of the unit weight of the soil.

In order to detect any significant variations in moisture content, numerous moisture content determinations were made during compaction. After the samples had been compacted, the weight of the soil, mold and base was determined to the nearest gram. Thus it was possible to calculate the initial dry weight of each of the samples.

The tops of the molds were then covered with

cellophane to prevent any evaporation. Over this was placed a thin circular plate which was held in position by tacks driven into the soil. The nuts on the uprights were adjusted to provide sufficient movement for the device which was used to measure the changes in the height of the sample. This device consisted of an Ames dial connected to a cross-bar, which could be set on the nuts on the uprights of each mold. (Plate 6 shows the general set-up except that this specimen was subjected to several cycles and approximately an inch of expansion has occurred. It may also be noted that the plate has been tilted due to differential movement of the soil).

The initial dial reading was measured and recorded to the nearest 1/10,000 of an inch and the cross-bar was removed. The initial weight of the specimen (mold, base plate, circular plate and soil) was measured and recorded to the nearest gram.

Cycling Procedure

The subgrade soils were subjected to 9 1/3 cycles. Each cycle consisted of three stages; saturation by capillarity* (1/3 cycle), freezing (2/3 cycle) and thawing (1 cycle). One specimen was tested for strength and water content at the end of each stage of saturation by capillarity up to the 4 1/3 cycle. The sixth mold was cycled and tested at the 9 1/3 cycle. This provided a coverage of the changes in strength, water content and density, through 9 1/3 cycles. (Appendix A)

* The degree of saturation existing when the soil has taken up all the possible water due to the capillary forces.

The granular material was subjected to six cycles, each cycle being the same as for the subgrade soil. The specimens were tested at the thawed (full cycle) and at the saturation by capillarity (1/3 cycle) stages of cycling. This provided a coverage of the changes in strength, water content and density through 6 cycles. (Appendix B)

The specimens were placed in water which was maintained at the bottom of the sample. Although no attempt was made to control the temperature of the water, it remained at approximately 20°C. The specimens were permitted to take up water by capillarity until they become saturated by capillarity. This was determined from the rates of change of the weights of the specimens. When the changes in weights became less then two grams per day the specimens were assumed to be saturated by capillarity.

When the specimens became saturated by capillarity, they were placed in the frost room (-30°C) for a period of 24 hours. The samples were frozen from all directions. On removal from the frost room the Ames dial readings were recorded. The specimens were permitted to thaw for a period of 24 hours at room temperature after which time the dial readings were recorded. In addition, the weight of the specimen was determined to detect any change in water content which may have occurred during the freeze-thaw period.

Determination of Strength and Water Content for Subgrade Soil

The strength of the subgrade soil was determined by the 1.4 inch diameter unconfined compression test. Since the samples were to be obtained with 1.4 inch sampling tubes, an investigation was carried out to determine whether to use a sampler wall thickness of 0.04 or 0.06 inch. The sampler with the 0.04 inch wall thickness showed less distrubance on samples but was impractical due to the deformation of the sampler as it was driven into the soil. Thus, it was decided to use the sampler with a wall thickness of 0.06 inch. A minimum clearance ratio* of four percent was maintained for each of the samplers, which were cleaned and greased before being driven into the soil.

Each of the specimens were tested as follows: After the thaw period two 1.4 inch samplers, approximately 6 inches long, were driven into the soil in the mold. The ends of the samplers were cleaned out and sealed with wax to prevent any entrance or escape of water. The specimen, along with the sealed samplers, was placed in water to permit the remainder of the sample to become saturated by capillarity. Three more 1.4 inch samplers were then evenly spaced around the mold and the sample was pushed out of the mold into the samplers. Moisture content determinations were made at the top, middle and bottom of the center of the soil specimen. The average moisture content was determined from the remaining soil. (Plate 31)

* Clearance Ratio (Inside)

$$= \frac{\text{Inside dia. of sampling tube} - \text{dia. of cutting edge}}{\text{Diameter of cutting edge}}$$

The unconfined compression test was carried out in the moisture room, in which the relative humidity was maintained close to 100 percent. Initial tests showed that a wide variation of strengths would be obtained if the unconfined tests were carried out at room humidity. To obtain the most representative sample for the height of the mold, the length of the unconfined samples were taken as long as would be permitted without the occurance of bending during the unconfined compression test. The unconfined test was carried out in accordance with the ASTM procedure as submitted by Housel[21]. Moisture contents were determined from the top, middle and bottom of the sample on the completion of the test.

Determination of Strength and Water Content for Granular Material

The strength of the granular material was evaluated by a multi-stage vacuum triaxial test (Plate 7). At the cycle for which the strength and the water content were desired, the soil sample was removed from the mold with as little disturbance as possible. It was immediately placed in the triaxial apparatus and the rubber membrane was pulled up over the sample to prevent any loss of moisture due to evaporation. The membrane was securely bound to the upper and lower loading plates. The extensometer, by which the vertical deflection was measured was adjusted to obtain the initial reading. In order to obtain the area of the sample

at failure, a circumference gauge was placed around the middle of the sample and maintained in this position throughout the test. The sample was placed so that the axial load could be applied through the loading beam of the platform scale, which was balanced for zero load.

A vacuum was applied to the specimen to check for any leakage through the membrane. When no leakage occurred, initial readings were recorded and the first stage of the test was undertaken. This was carried out under zero confining pressure. The loads were applied at minute intervals with the extensometer readings being recorded on the half minute. The magnitude of the loads applied depended on the expected failure load. As the sample approached failure conditions, the load increments were greatly reduced. Failure conditions were assumed to exist when the rate of yielding of the specimen became excessive or if the load could not be immediately picked up by the sample. For the last increment of loading, the perimeter of the sample was recorded from the circumference gauge.

At failure, the load was removed and the confining pressure increased to 0.2 atmospheres. The procedure of stage one was then repeated. After similar stages of 0.4, 0.6 and 0.9 atmospheres pressure, the sample was again tested under zero confining pressure to check on the amount of disturbance.

On the completion of the test, the apparatus was disassembled and moisture contents were determined for the top and bottom half of the sample.

## SUMMARY OF SOIL IDENTIFICATION TESTS

| Sample | Atterberg Limits | | | Specific Gravity | Standard Proctor | | Modified Proctor | | Grain Sizes | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | $W_1$* | $W_p$* | $I_p$* | | $\gamma_d$* | Opt. w* | $\gamma_d$* | Opt. w | $D_{10}$mm* | $D_{60}$mm* | Cu* |
| CL | 23.5% | 18.1% | 5.4% | 2.73 | 116.0 | 13.6% | 125.0 | 11.0% | 0.0008 | 0.18 | 225 |
| CI (1) | 35.8 | 17.5 | 18.3 | 2.77 | 110.5 | 14.0 | 125.8 | 9.5 | 0.0014 | 0.11 | 79 |
| CI (2) | 48.2 | 21.6 | 26.6 | 2.79 | 104.8 | 18.0 | 120.0 | 12.5 | 0.0015 | 0.022 | 14.[illegible] |
| CH | 68.7 | 28.6 | 40.1 | 2.81 | 89.3 | 20.5 | 104.0 | 20.0 | 0.0040 | 0.001 | 2.[illegible] |
| Gravel | 40.9 | 19.9 | 21.0 | 2.68 | 119.0 | 7.8 | 133.0 | 7.4 | 0.10 | 3.1 | 31 |

* $W_1$ - Liquid Limit
$W_p$ - Plastic Limit
$I_p$ - Plastic Index
$\gamma_d$ - Dry Density (pounds/cu.ft.)
**Opt. w - Optimum water content**

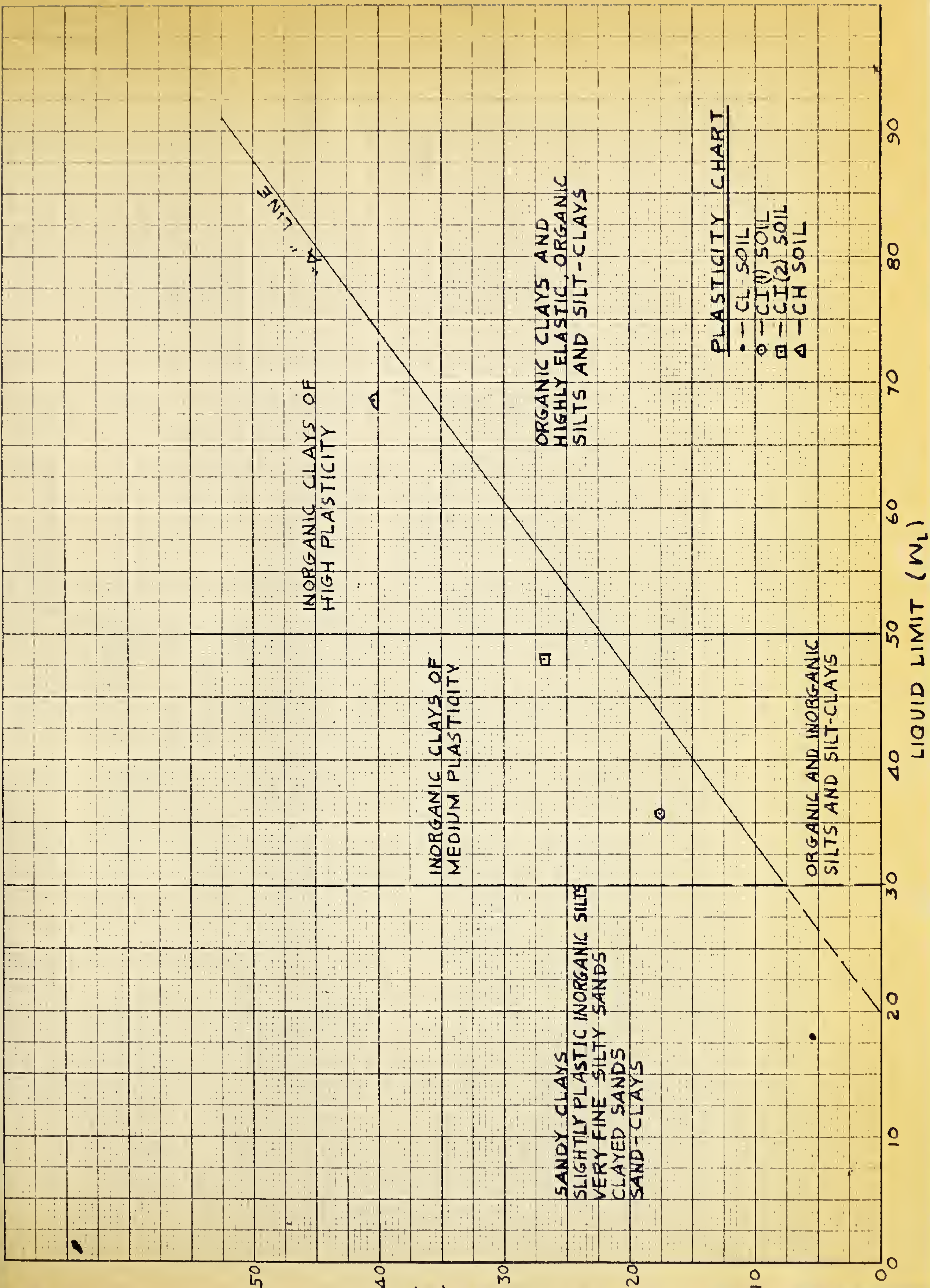
$D_{10}$ - Soil diameter of which 10% of the soil is finer
$D_{60}$ - Soil diameter of which 60% of the soil is finer
Cu - Coefficient of uniformity

PLATE 2

PLATE 2

PLATE 3

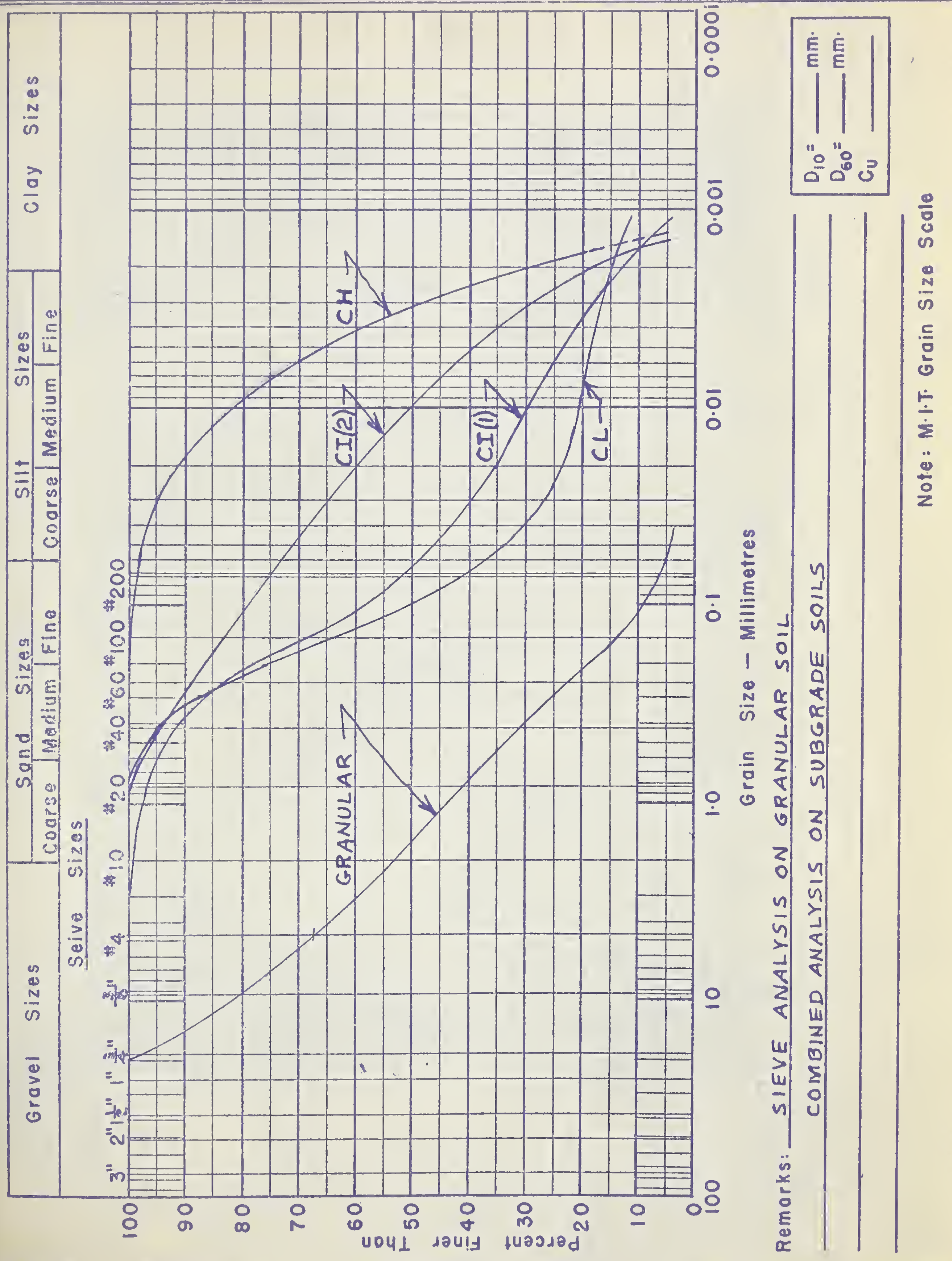


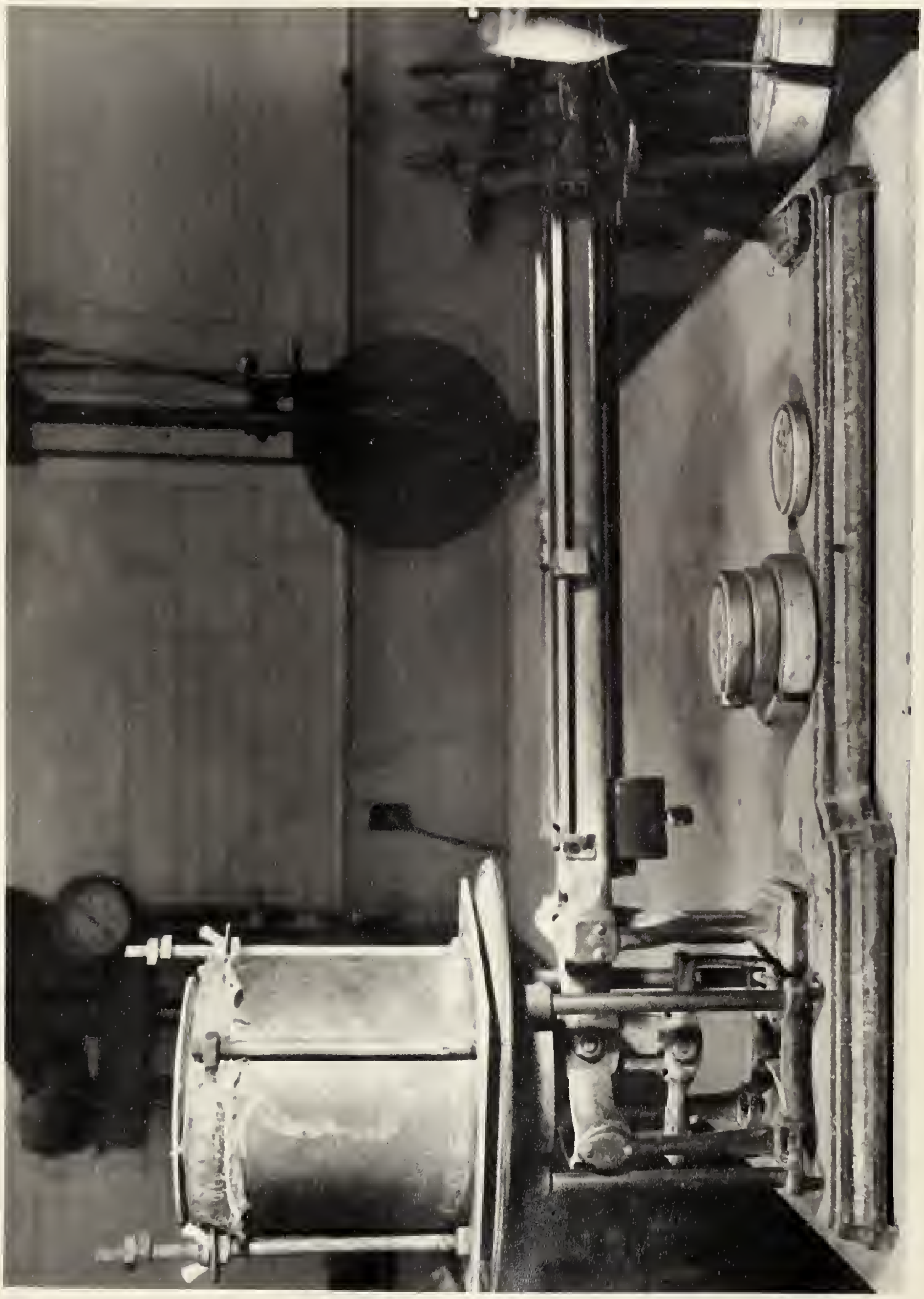

PLASTICITY CHART
• — CL SOIL
⊙ — CI(1) SOIL
⊡ — CI(2) SOIL
Δ — CH SOIL
"A" LINE
INORGANIC CLAYS OF HIGH PLASTICITY
ORGANIC CLAYS AND HIGHLY ELASTIC, ORGANIC SILTS AND SILT-CLAYS
INORGANIC CLAYS OF MEDIUM PLASTICITY
ORGANIC AND INORGANIC SILTS AND SILT-CLAYS
SANDY CLAYS
SLIGHTLY PLASTIC INORGANIC SILTS
VERY FINE SILTY SANDS
CLAYED SANDS
SAND-CLAYS
0 10 20 30 40 50 60 70 80 90
LIQUID LIMIT ($W_L$)
0 10 20 30 40 50
PLASTIC INDEX ($I_P$)

UNIVERSITY of ALBERTA
DEPT. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

## GRAIN SIZE CURVE

| | |
|---|---|
| PROJECT THESIS | |
| SITE | |
| SAMPLE | |
| LOCATION | |
| HOLE | DEPTH |
| TECHNICIAN | DATE |





WEIGHT DETERMINATION OF TYPICAL SPECIMEN

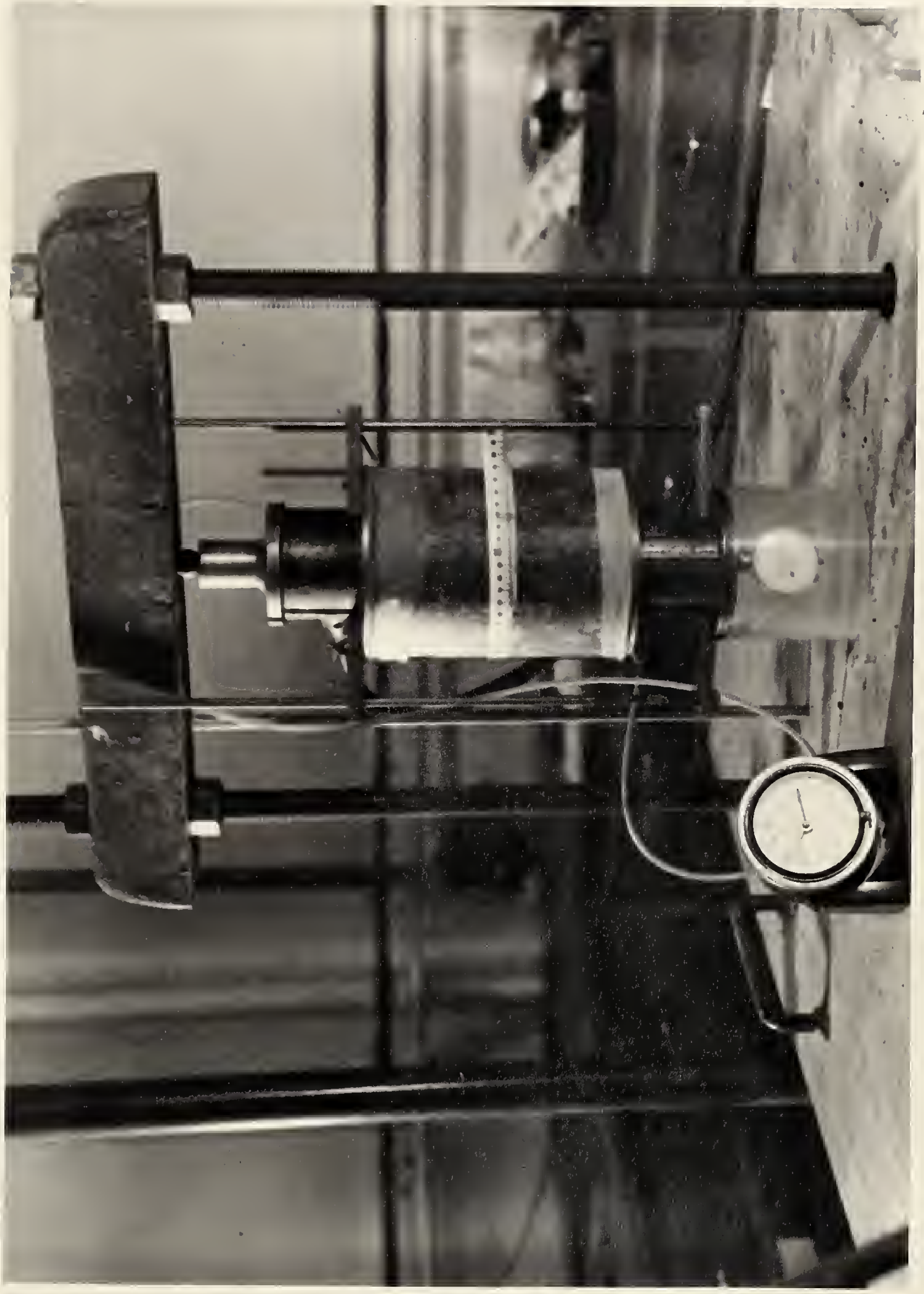
PLATE 5

PLATE 5

TYPICAL SPECIMEN AFTER SEVERAL CYCLES
(SHOWING AMES DIAL IN POSITION)

PLATE 6

PLATE 6

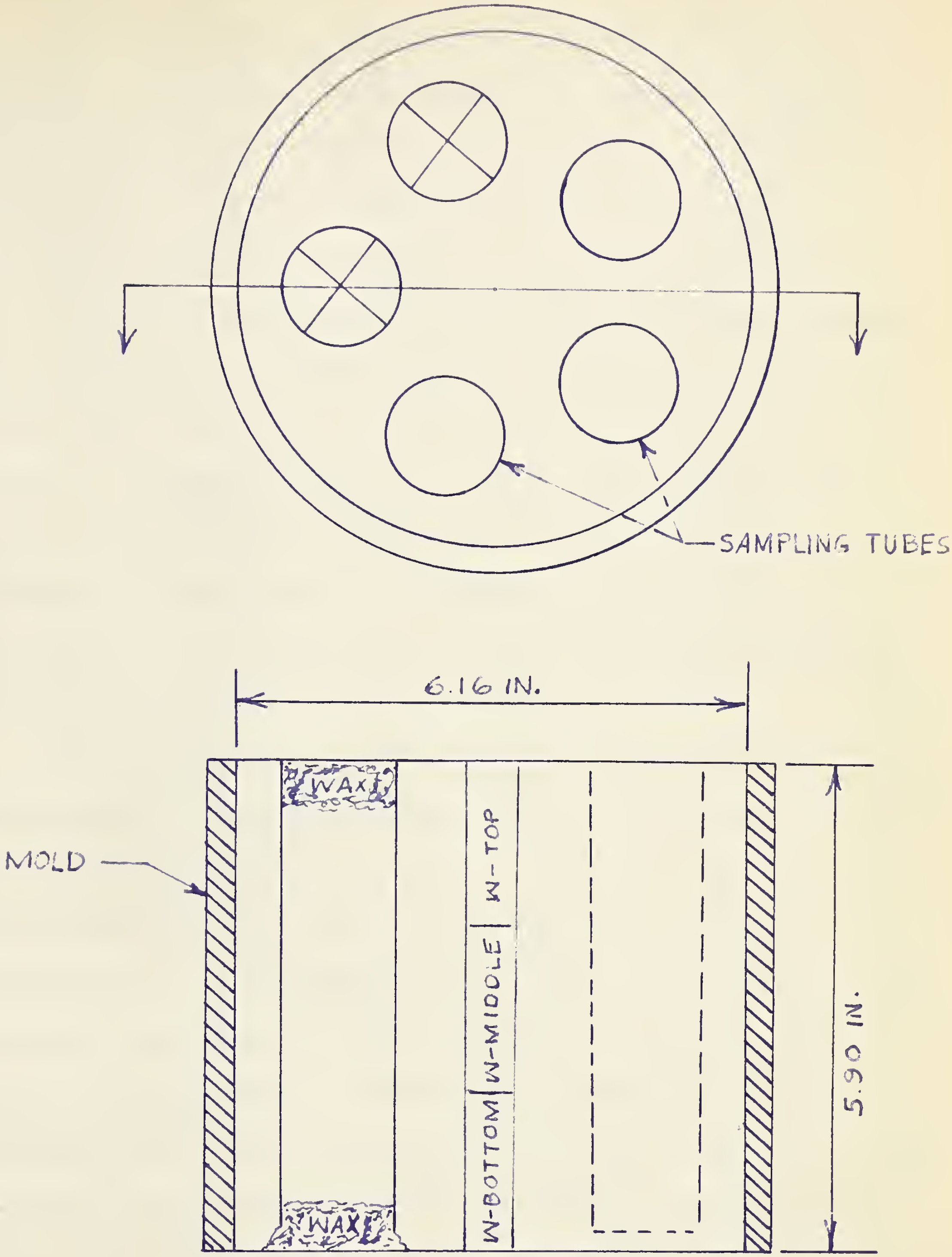


MULTI-STAGE VACUUM TRIAXIAL SET UP

PLATE 7

PLATE 7

PLATE 31




NOTE :- ⊗ - SEALED SAMPLERS
○ - UNSEALED SAMPLERS (SOIL DRIVEN OUT OF MOLD INTO SAMPLERS)

DIAGRAM OF SAMPLE SHOWING POSITION OF SAMPLERS

## CHAPTER IV - RESULTS

On compacting the soil in the molds for the test procedure, variations in the initial density up to one percent were found. This may be explained by the variation in the water content on compaction which in some cases was up to one percent. In order to compare the changes in density, the initial densities were averages for each series of tests. The average changes in density of the soils (Plate 8) were calculated for each stage of cycling. (Appendix A and B). Since these changes were based on the changes of the volume of the sample, as determined by the Ames dial readings, (Plate 6) it was impossible to determine the variation in density throughout the soil sample.

It was found that the initial densities of the subgrade materials were up to three percent less when the soil was compacted in the 6.16 inches in diameter molds than in the standard Proctor molds. This may be attributed to the difference in the diameter of the molds, which would exhibit different confining pressures, or to the volume occupied by the grease and cellophane, which was not considered in the determination of the unit weight. For the granular material, the densities obtained for the two series of tests were greater than those determined by the moisture-density relationship. Since the same molds and compaction procedure

were used in each case, this difference may be due to non-uniformity of the material or to errors in the determination of the weight of soil.

In order to obtain the most favourable relationship between the strength and the number of cycles for the subgrade soils, the following plots are presented:

Plate 9 - Average Water Content vs Average Strength.
The average water content is plotted against the average unconfined compression strength for each soil at the saturation by capillarity cycles of testing. (Appendix C).

Plate 10 - Average Void Ratio vs Average Strength.
The average void ratio is plotted against the average unconfined compression strength for each soil at the saturation by capillarity cycles of testing. (Appendix C).

Plate 11 - Average Degree of Saturation vs Average Strength.
The average degree of saturation is plotted against the average unconfined compression strength for each soil at the saturation by capillarity cycles of testing. (Appendix C).

Plate 12 - Water Content vs. Strength - CL Soil

Plate 13 - Water Content vs Strength CI (1) Soil

Plate 14 - Water Content vs Strength CI (2) Soil

Plate 15 - Water Content vs Strength CH Soil

Plates 12 to 15 are plotted on the basis of the maximum water content found in the unconfined compression test specimen, as was determined from the water content at the top, middle and bottom of the specimen. (Appendix A).

From Plates 9 to 15, it was found that the plots of water content vs strength provided the most obvious

relationship from which an analysis of the results could be carried out. In order to provide a definite relationship between water content and strength, it was assumed that the water content vs strength could be represented by a single curve drawn through the approximated average values. Thus, by determining the minimum and maximum water content in the mold, the minimum and maximum unconfined compression strength could be estimated for each stage of cycling.

The variation of water content for the number of cycles, strength and density for the subgrade soils are presented on the following plates:

Plate 16 - CL Soil Changes, Standard Proctor Density Series
Plate 17 - CL Soil Changes, Modified " " "
Plate 18 - CI (1) Soil Changes, Standard Proctor Density Series
Plate 19 - CI (1) Soil Changes, Modified " " "
Plate 20 - CI (2) Soil Changes, Standard " " "
Plate 21 - CI (2) Soil Changes, Modified " " "
Plate 22 - CH Soil Changes, Standard Proctor Density Series
Plate 23 - CH Soil Changes, Modified " " "

The plots of the number of cycles against water content were based on the minimum and maximum water content found in the sample at the saturation by capillarity stages (the 1/3 stage of each cycle of testing). (Appendix A). The unconfined compression test specimens which were sealed at the thawed stage of cycling were not considered due to the possible re-distribution of water within the samples. Thus, the two extreme conditions of water content were determined for each of the cycles. The average water content was determined at each stage of saturation by capillarity from the

increase in the weight of the water in the soil, as it was compacted in the mold. (Appendix C). In several instances, particularly in the CL soil-modified Proctor density series, (Plate 17), the average water content as determined from the increase in the weight of water in the soil, as it was compacted in the mold was found to be higher than the maximum value as determined by the actual moisture content determinations throughout the samples. This may be due to the consolidation effect of the sampling tubes as they were being driven into the soil to be sealed in the molds. (Plates 16 to 29).

The plots of strength vs water content (Plates 16 to 29) were transferred directly from the plots of strength vs water content (Plates 12 to 15) for each of the respective soils.

On the plot of density vs water content, the standard and modified Proctor density curves are shown for each of the respective soils, along with the 100 percent saturation curves. The variation in the water content for the average density at each stage of saturation by capillarity was determined from the plot of cycles vs water content. This was presented to show the variation in water contents which could exist for the average density at each stage of cycling.

The results of the strength tests on the granular soil are shown on Plate 24. These are presented in the form of major principal stress at failure against the number of cycles for each minor principal stress. The

curves shown on Plate 24 are drawn through the points of equal minor principal stresses.

The variation of water content for the number of cycles, compressive stress and density for the granular soil are shown on Plates 25 and 26. The plots of water content against cycles and density were determined by the same method as that outlined for plates 16 to 23. The plot of compressive stress vs water content was obtained from Plate 24, by determining the water content for the number of cycles from the plot of cycles vs water content.

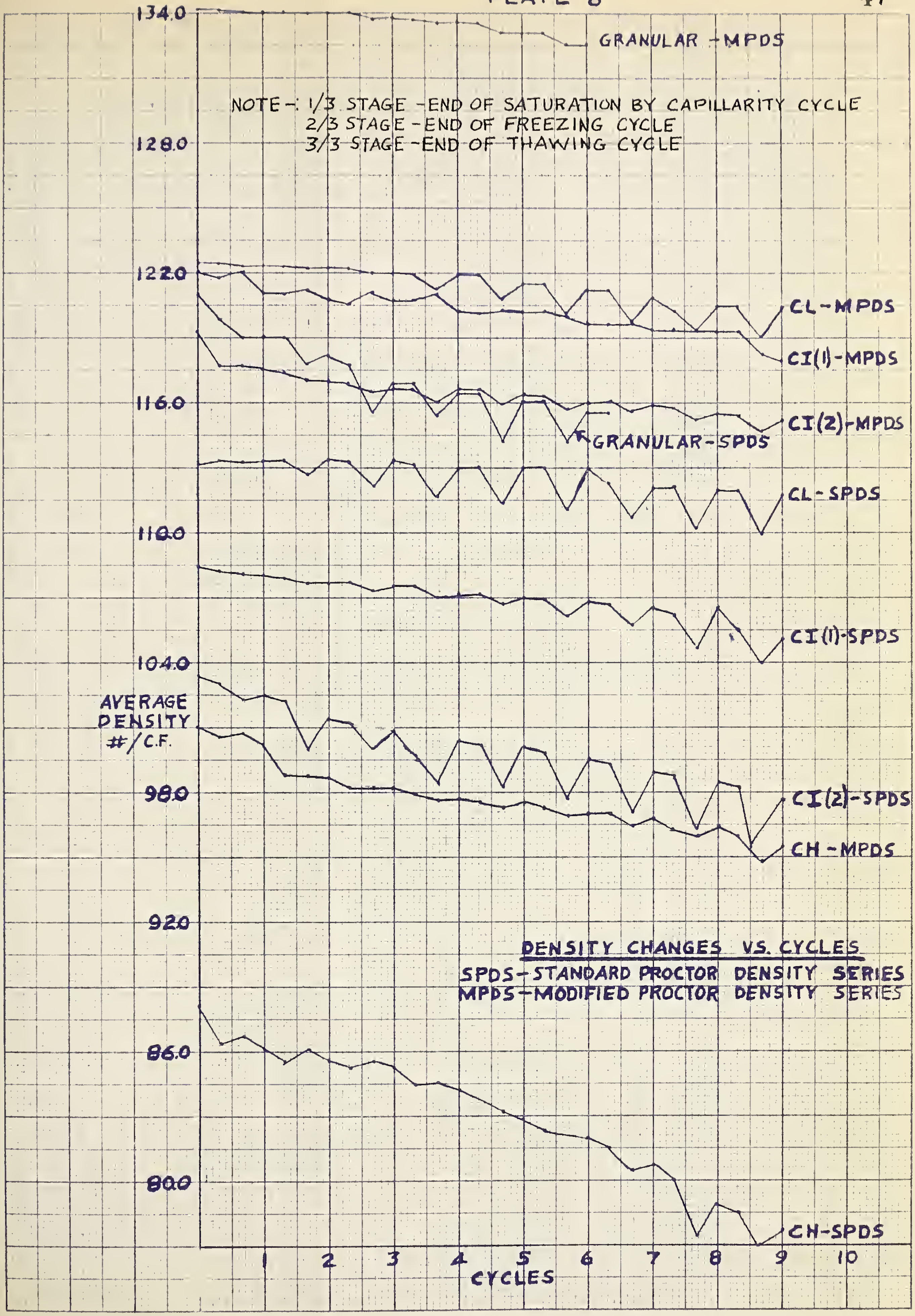
The stress strain relations for the granular soil are presented on the following Plates:

Plate 27 - Granular Soil, Standard Proctor Density Series
Plates 28 and 29 - Granular Soil, Modified Proctor Density Series.

In each case, the strains are based on the accumulative strains on the specimen. The density of the specimen ($\gamma_d$) at the end of each stage of testing was calculated from the changes in volume. The volume of the sample was approximated by multiplying the height of the specimens by the average of the end and middle areas. This would involve an error which would tend to give values of density greater than the actual density, due to the curvature of the specimen. The error introduced due to the measurements was in the order of $\pm$ 0.2 percent and would probably be insignificant as

compared to that portion of the volume which was neglected due to the curvature of the specimen. Thus, the degree of accuracy to which the densities were determined could not be obtained.

PLATE 8



GRANULAR - MPDS
NOTE -: 1/3 STAGE - END OF SATURATION BY CAPILLARITY CYCLE
2/3 STAGE - END OF FREEZING CYCLE
3/3 STAGE - END OF THAWING CYCLE
134.0
128.0
122.0
116.0
110.0
104.0
98.0
92.0
86.0
80.0
AVERAGE DENSITY #/C.F.
CL - MPDS
CI(1) - MPDS
CI(2) - MPDS
GRANULAR - SPDS
CL - SPDS
CI(1) - SPDS
CI(2) - SPDS
CH - MPDS
CH - SPDS
1
2
3
4
5
6
7
8
9
10
CYCLES
DENSITY CHANGES VS. CYCLES
SPDS - STANDARD PROCTOR DENSITY SERIES
MPDS - MODIFIED PROCTOR DENSITY SERIES

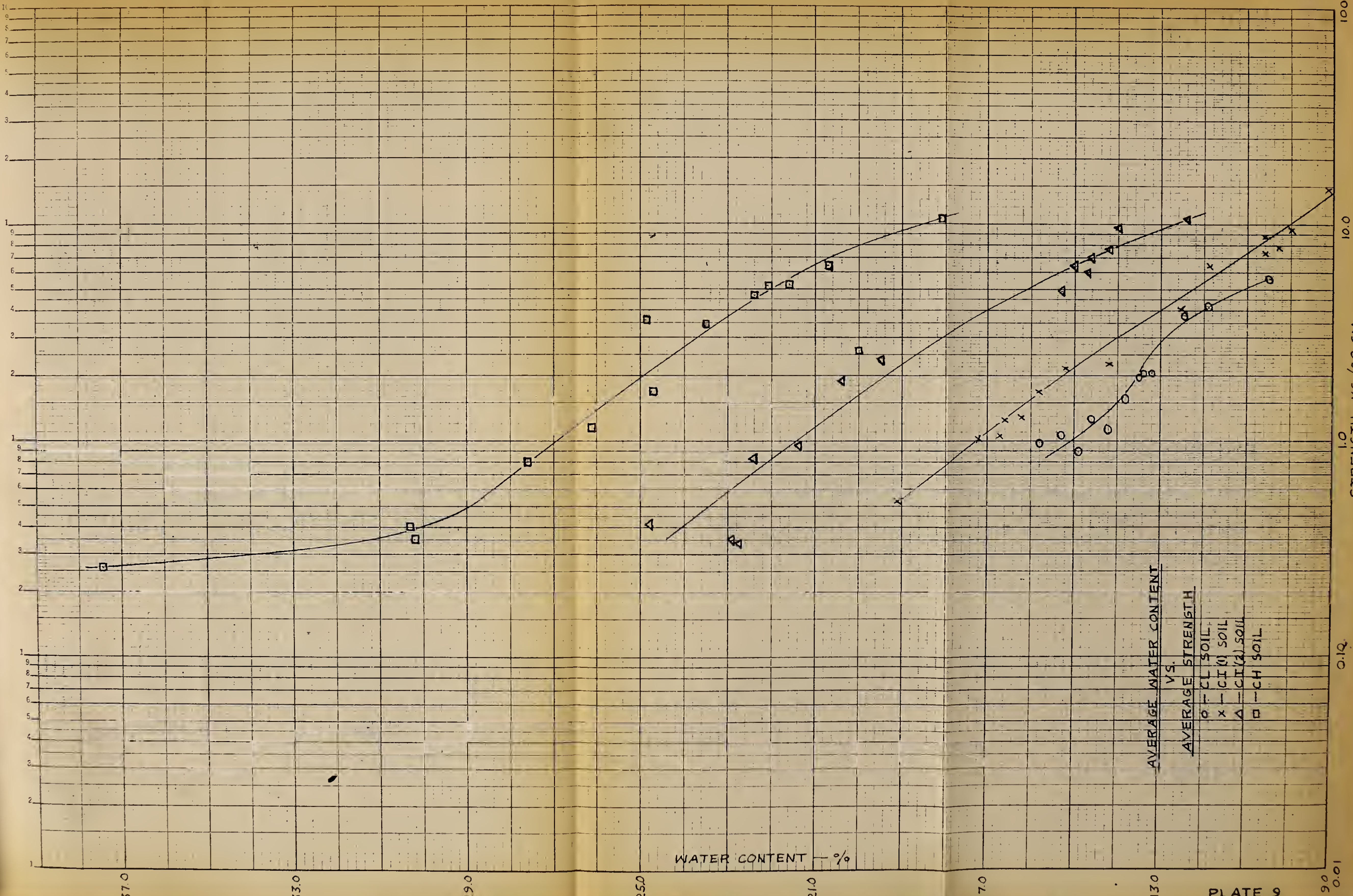
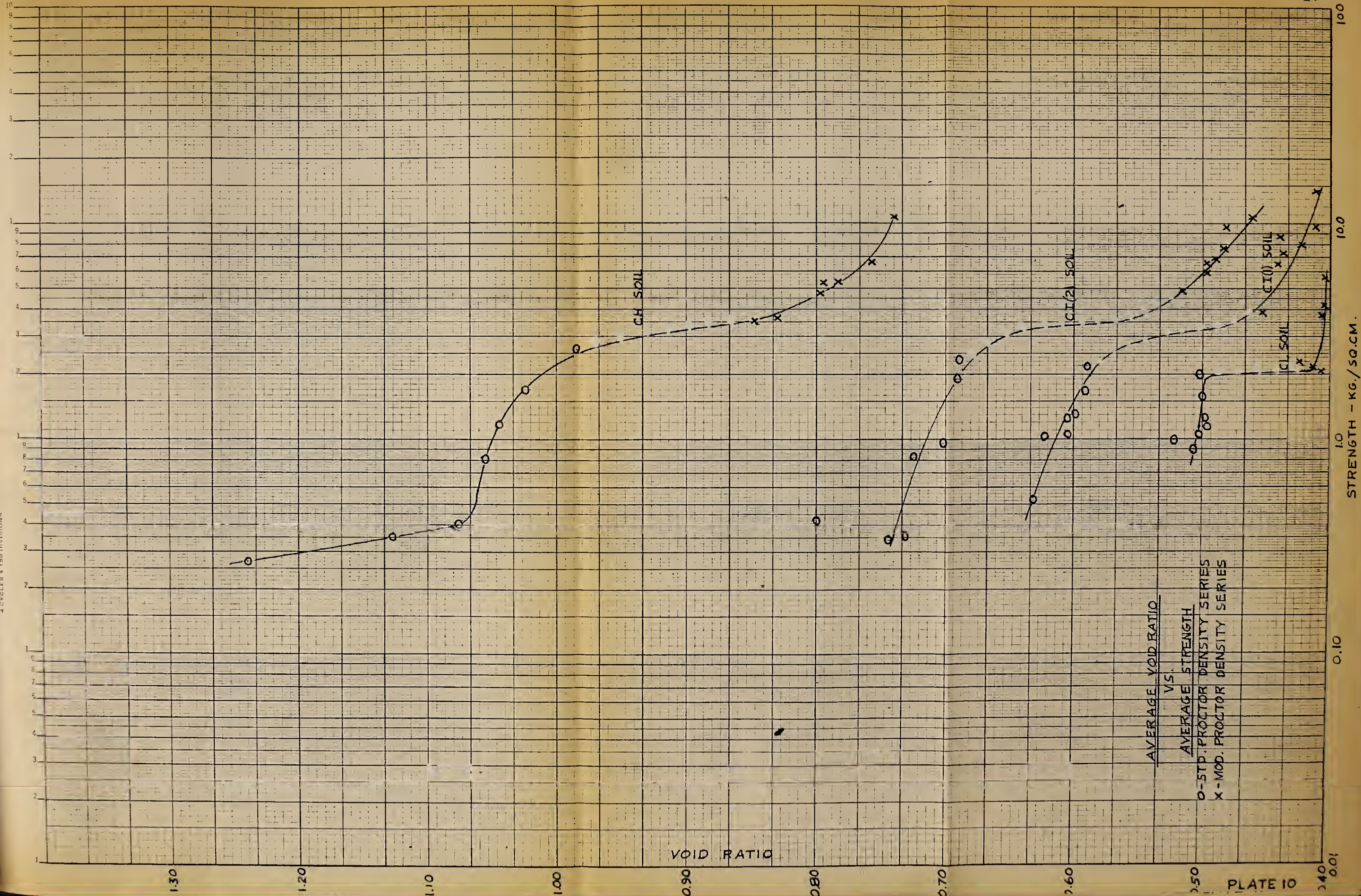
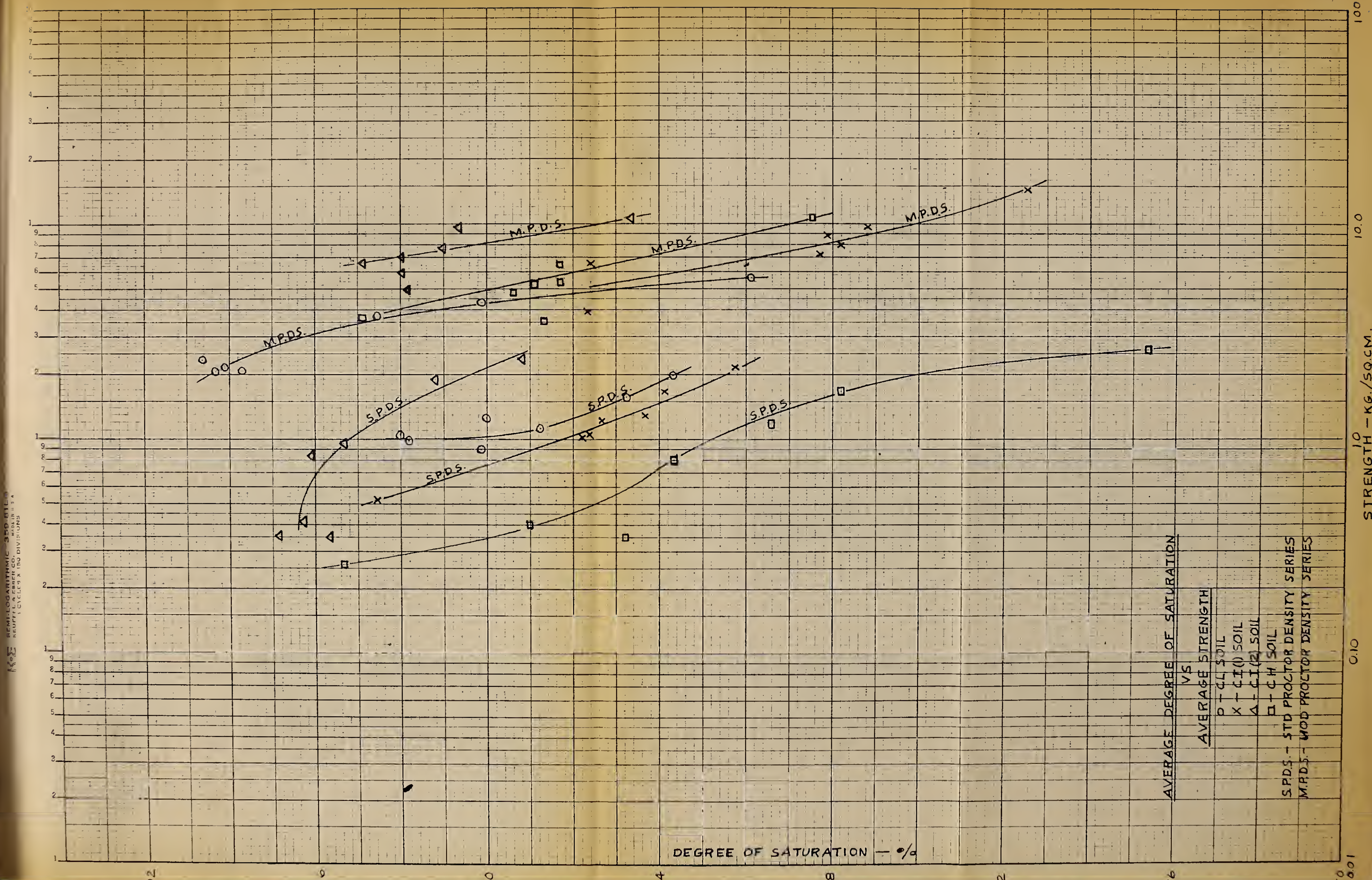

AVERAGE WATER CONTENT
VS.
AVERAGE STRENGTH
o — CL SOIL
x — CI (1) SOIL
Δ — CI (2) SOIL
□ — CH SOIL
WATER CONTENT — %
STRENGTH - KG./SQ.CM
37.0
33.0
29.0
25.0
21.0
17.0
13.0
9.0
0.01
0.10
1.0
10.0
100

PLATE 10

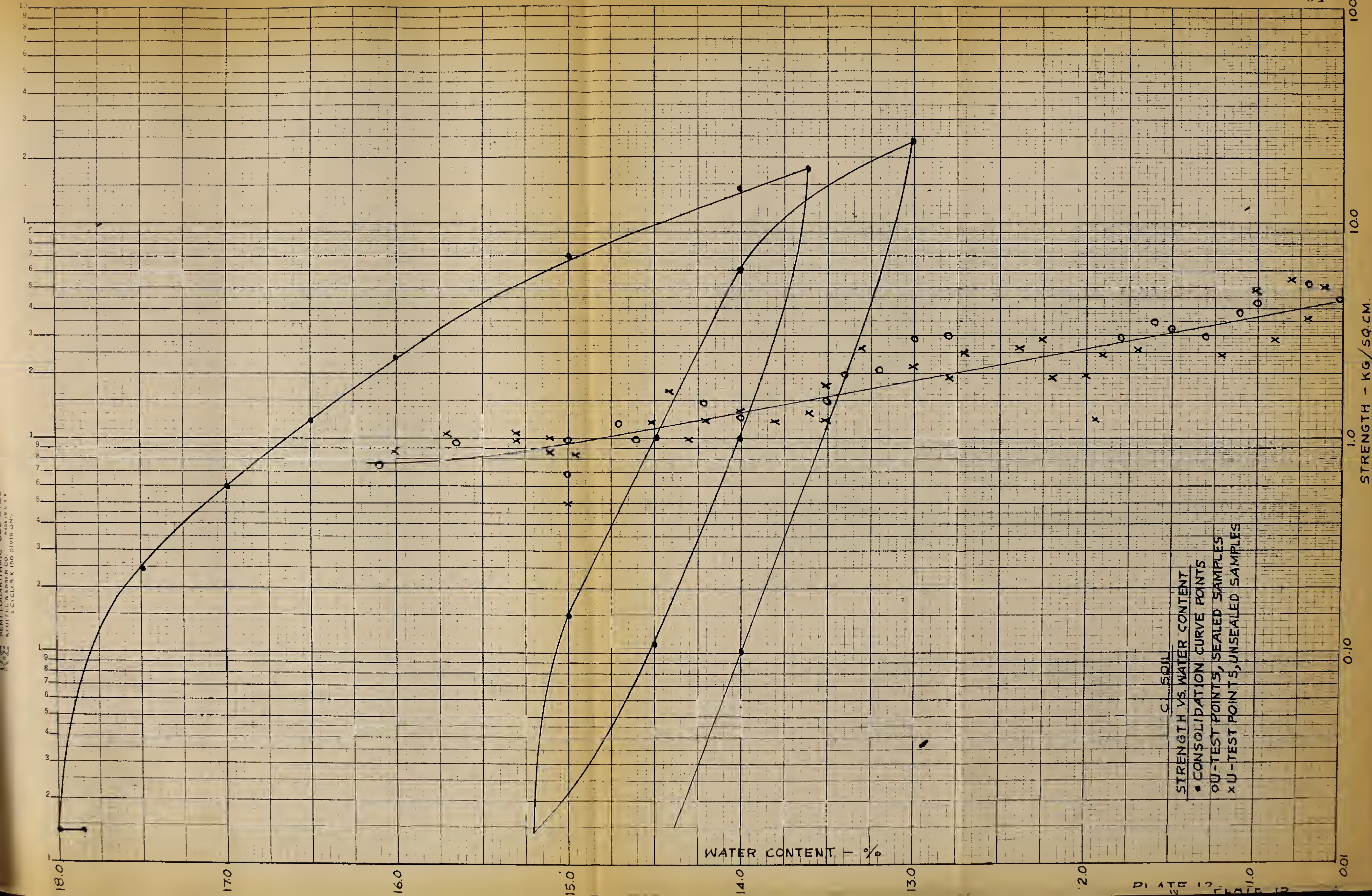



CH SOIL
CI(2) SOIL
CI(1) SOIL
CL SOIL
AVERAGE VOID RATIO
VS.
AVERAGE STRENGTH
O-STD. PROCTOR DENSITY SERIES
X-MOD. PROCTOR DENSITY SERIES
VOID RATIO
1.30
1.20
1.10
1.00
0.90
0.80
0.70
0.60
0.50
0.40
STRENGTH - KG./SQ.CM.
100
10.0
1.0
0.10
0.01
PLATE 10



AVERAGE DEGREE OF SATURATION
VS
AVERAGE STRENGTH
○ - C.L. SOIL
X - C.I.(1) SOIL
△ - C.I.(2) SOIL
□ - C.H. SOIL
S.P.D.S. - STD PROCTOR DENSITY SERIES
M.P.D.S. - MOD PROCTOR DENSITY SERIES
M.P.D.S.
S.P.D.S.
STRENGTH - KG./SQ.CM.
DEGREE OF SATURATION — %
92
86
80
74
68
62
56
50
100
10.0
1.0
0.10
0.01
SEMI-LOGARITHMIC 359-81LG
KEUFFEL & ESSER CO.


PLATE II


C SOIL
STRENGTH VS. WATER CONTENT
• CONSOLIDATION CURVE POINTS
o U-TEST POINTS, SEALED SAMPLES
x U-TEST POINTS, UNSEALED SAMPLES
WATER CONTENT - %
STRENGTH - KG/SQ.CM.
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
100
10.0
1.0
0.10
0.01

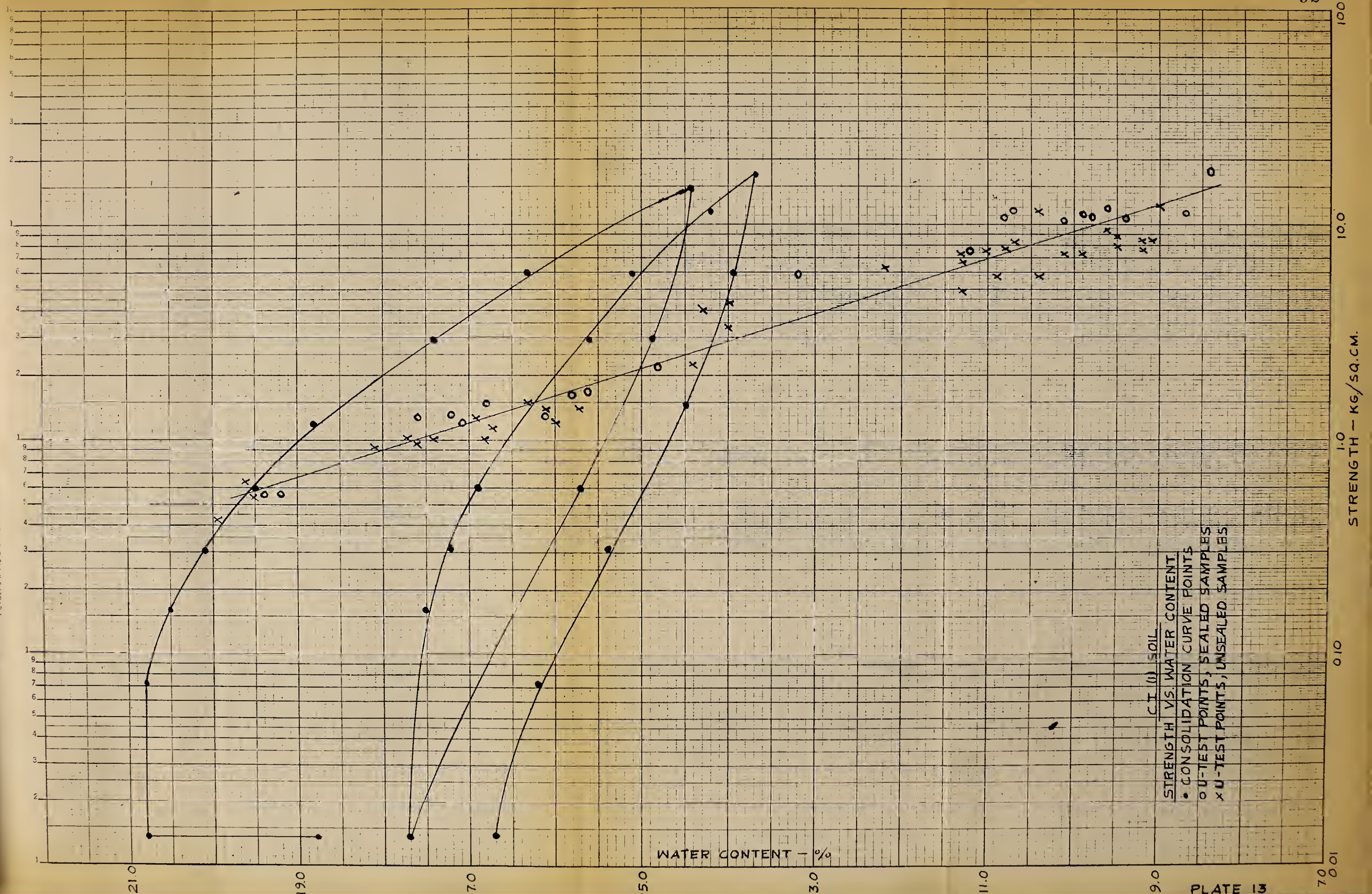

C I (1) SOIL
STRENGTH VS. WATER CONTENT
• CONSOLIDATION CURVE POINTS
o U-TEST POINTS, SEALED SAMPLES
x U-TEST POINTS, UNSEALED SAMPLES
WATER CONTENT − %
STRENGTH − KG/SQ.CM.
21.0
19.0
17.0
15.0
13.0
11.0
9.0
7.0
100
10.0
1.0
0.10
0.01

PLATE 13

PLATE 14

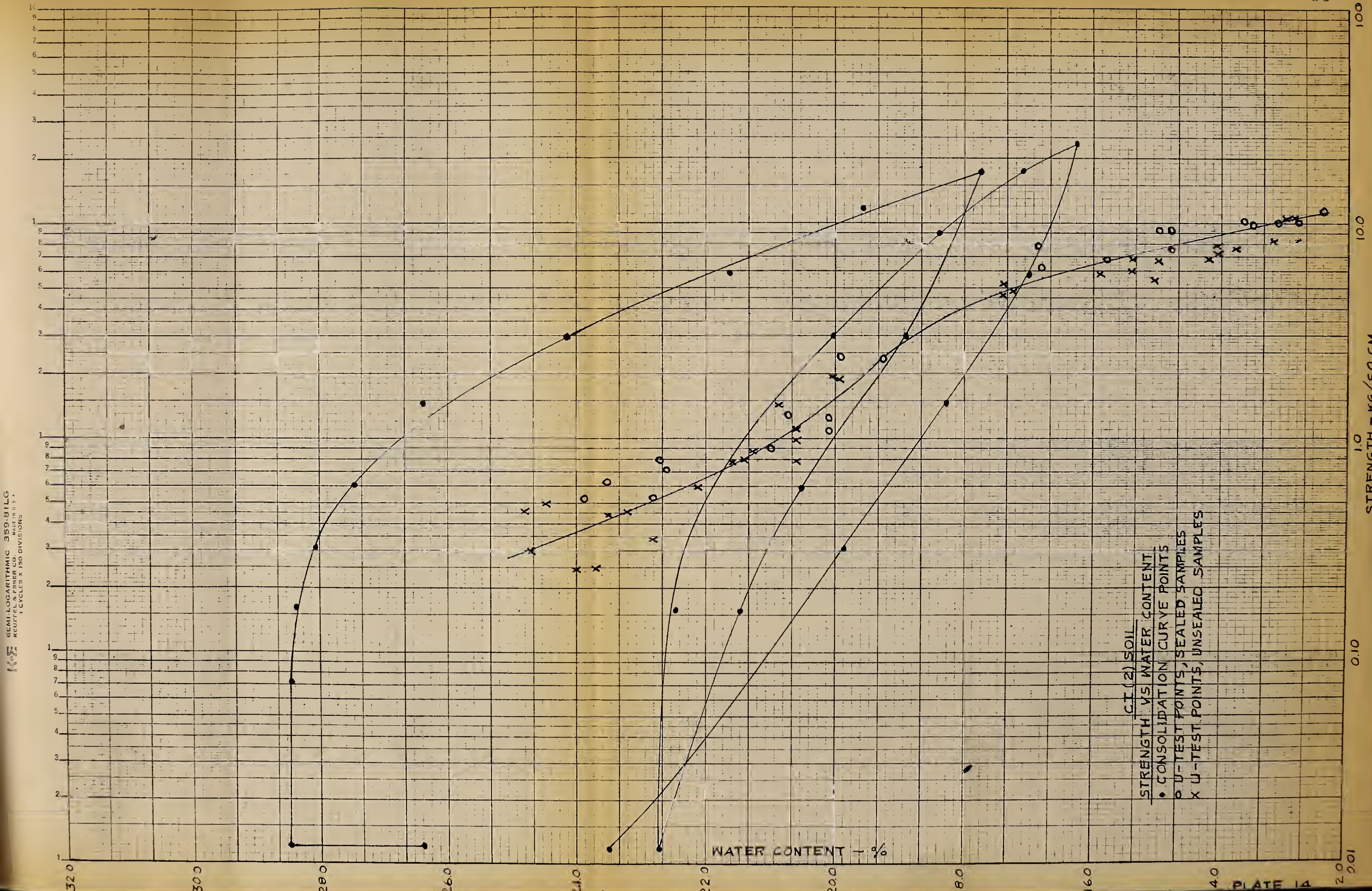
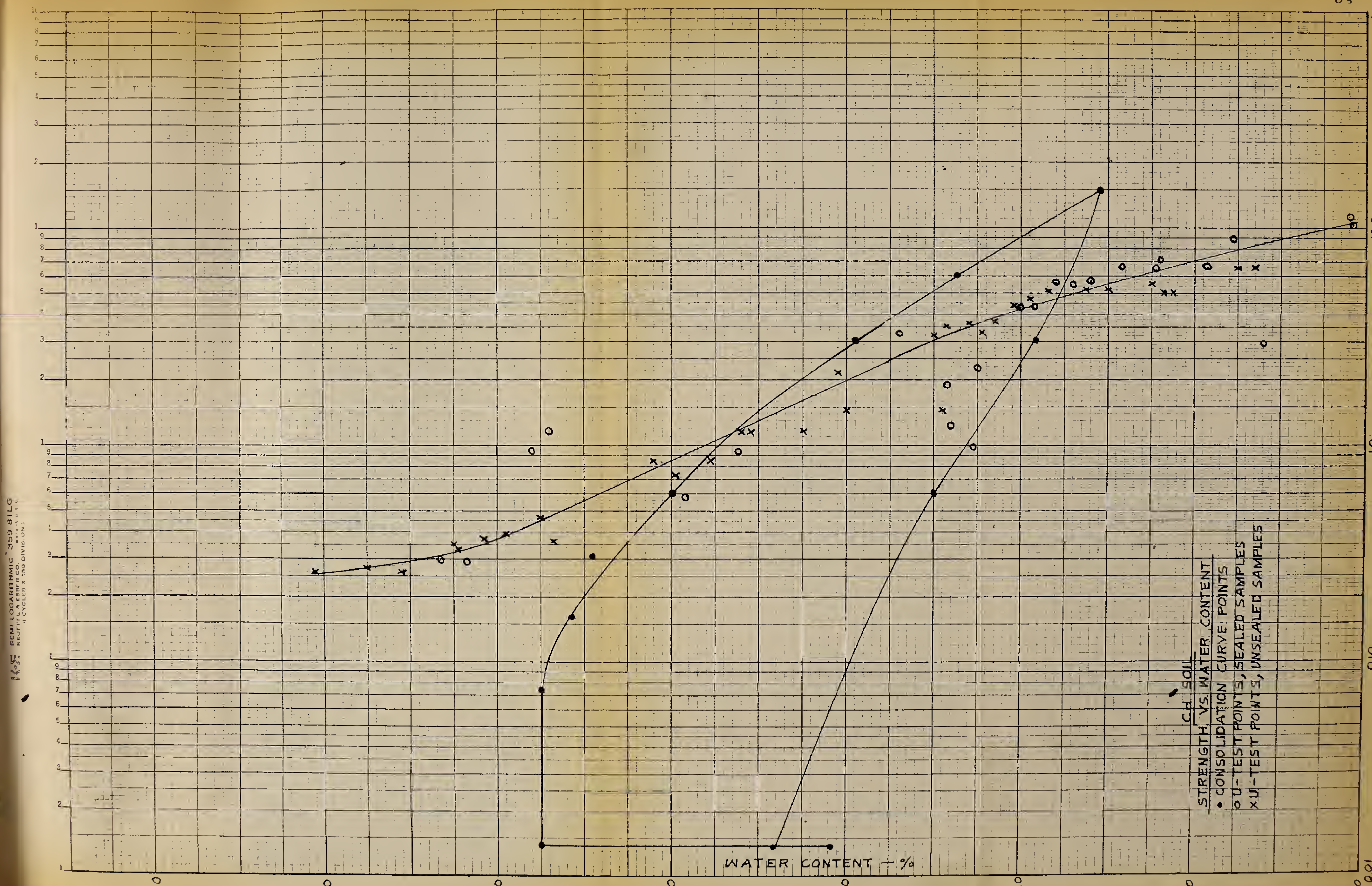
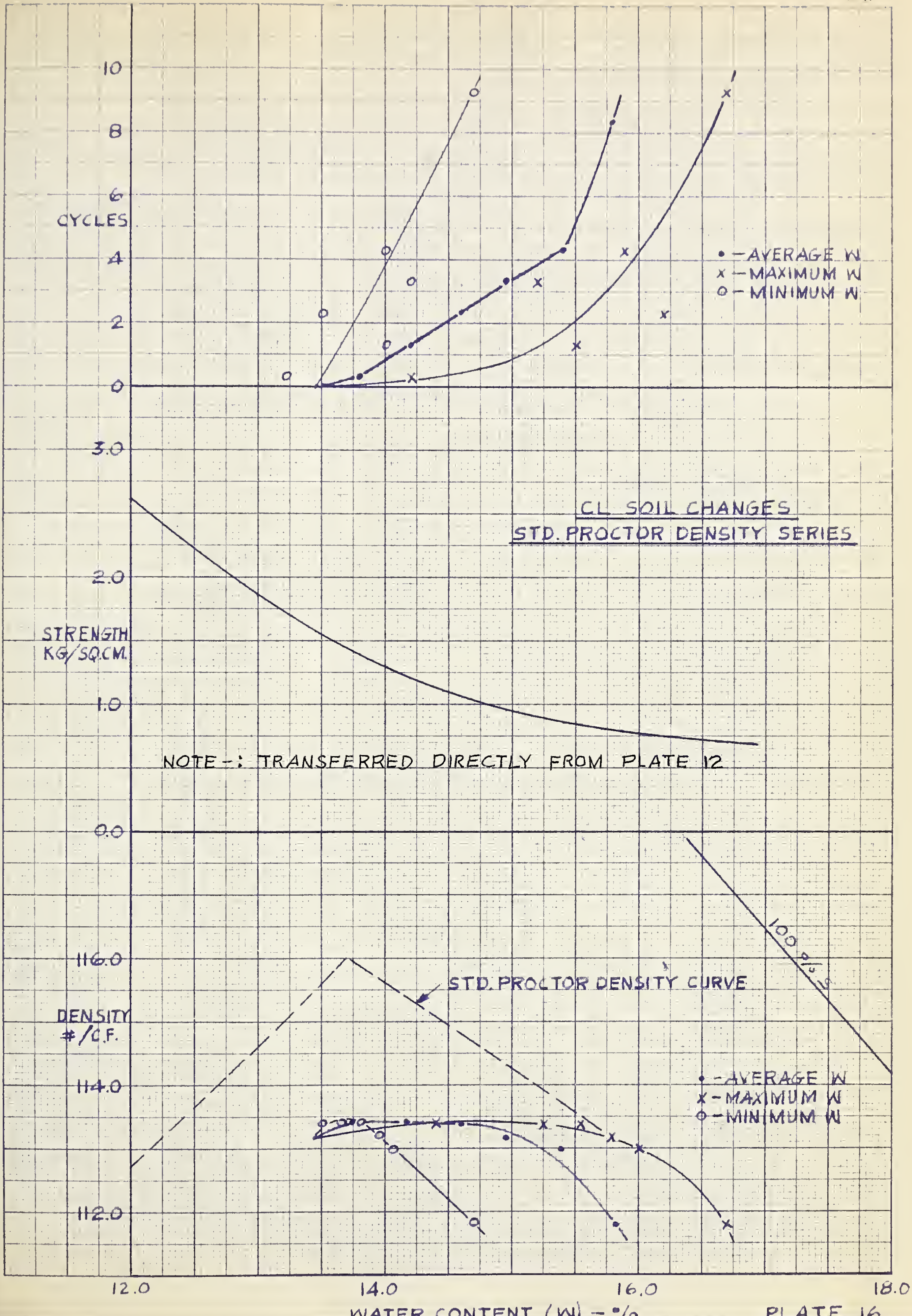
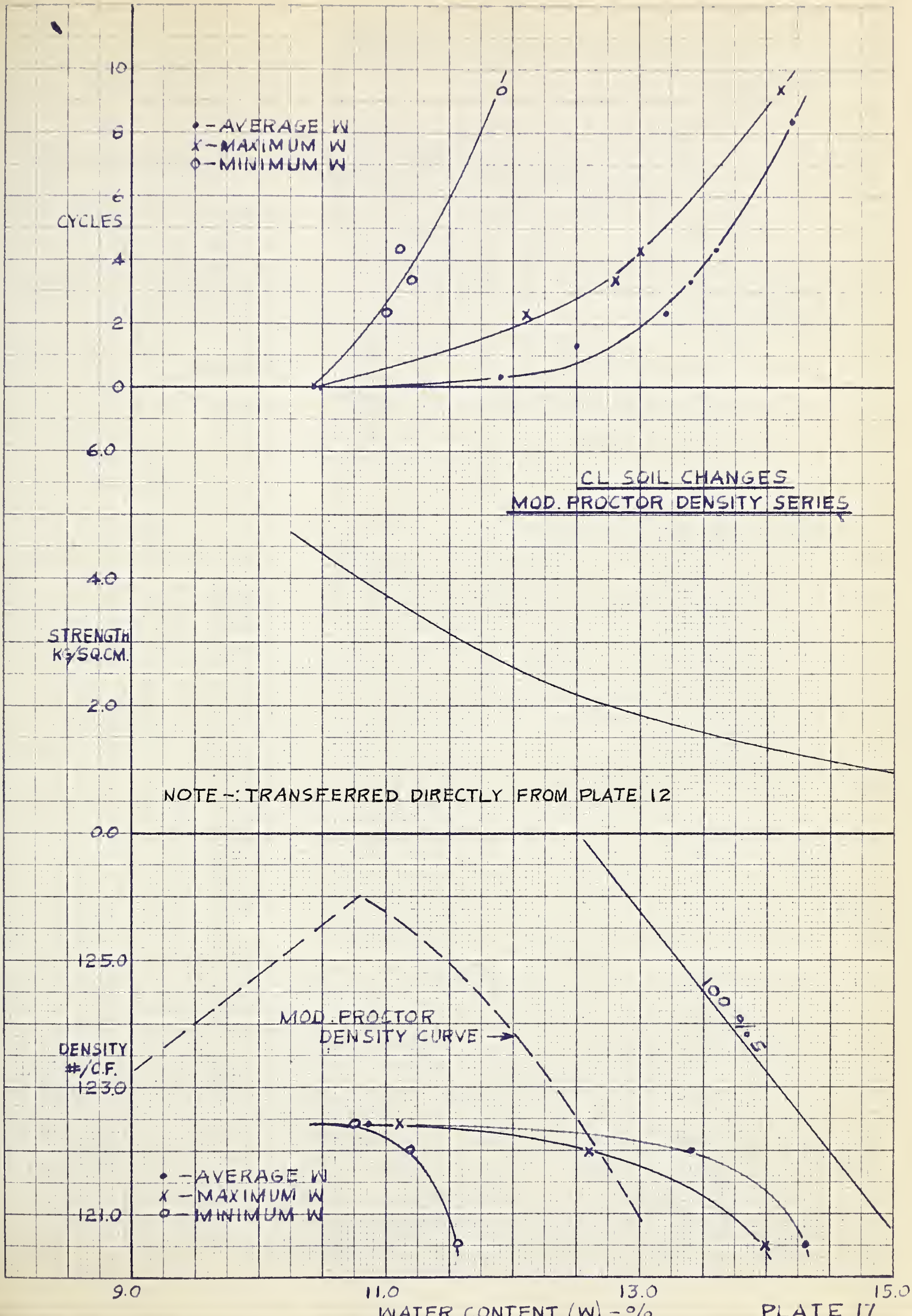


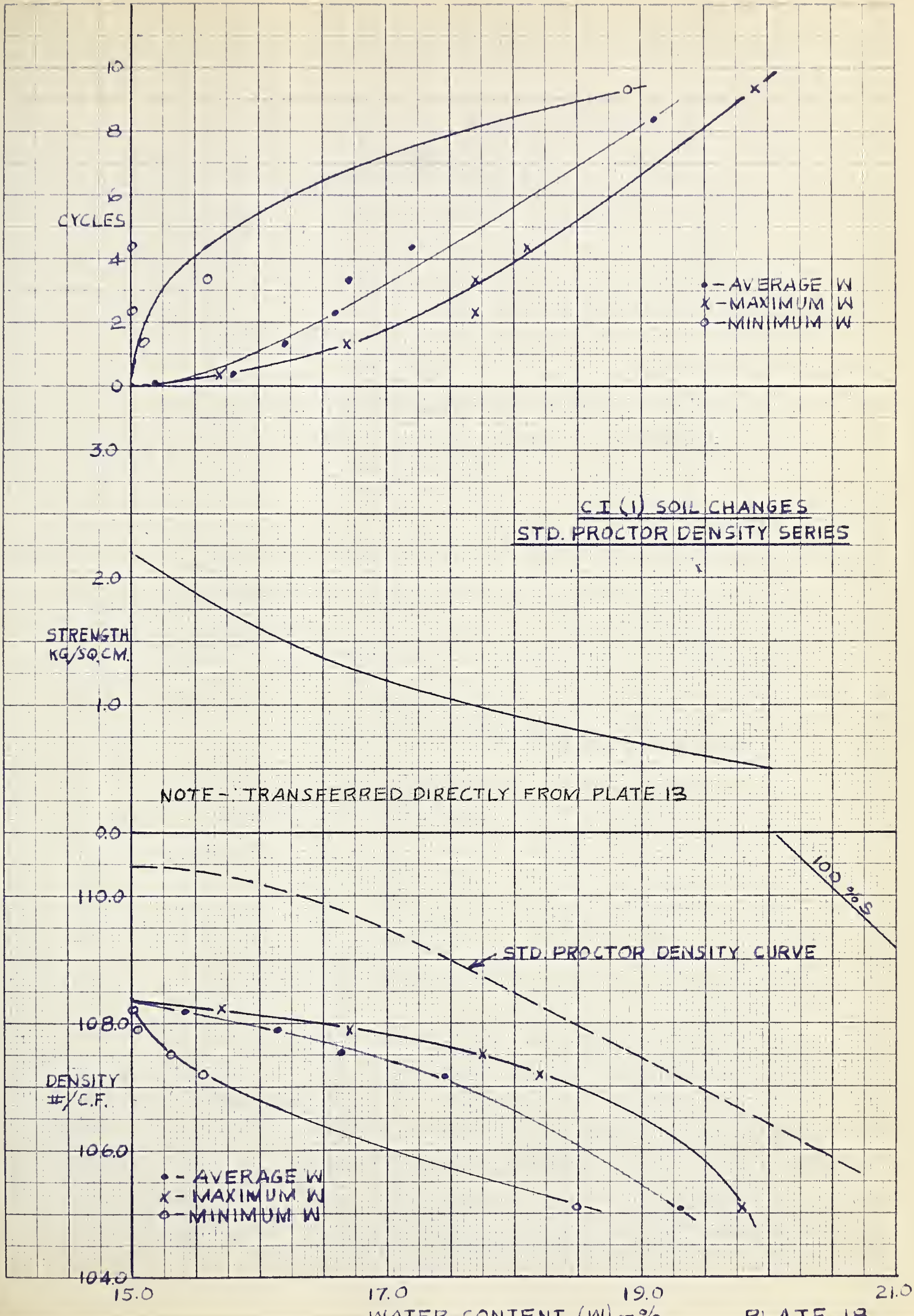

PLATE 14

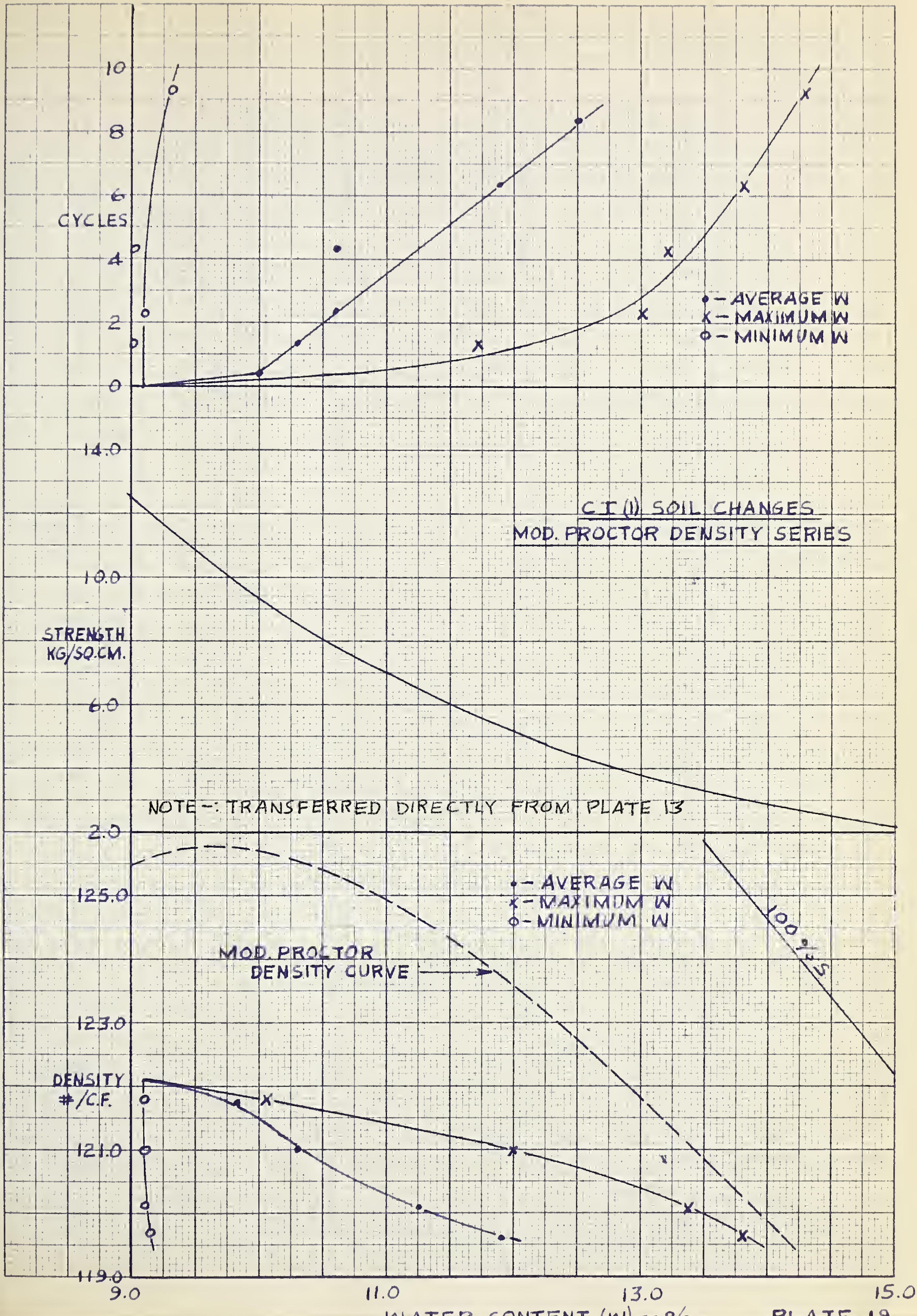

CH SOIL
STRENGTH VS. WATER CONTENT
• CONSOLIDATION CURVE POINTS
o U-TEST POINTS, SEALED SAMPLES
x U-TEST POINTS, UNSEALED SAMPLES
WATER CONTENT — %
STRENGTH - KG/SQ.CM.
45.0
41.0
37.0
33.0
29.0
25.0
21.0
17.0
0.01
0.10
1.0
10.0
100


CYCLES
10
8
6
4
2
0
• — AVERAGE W
x — MAXIMUM W
o — MINIMUM W
CL SOIL CHANGES
STD. PROCTOR DENSITY SERIES
3.0
2.0
STRENGTH KG/SQ.CM.
1.0
NOTE —: TRANSFERRED DIRECTLY FROM PLATE 12
0.0
100 %
116.0
STD. PROCTOR DENSITY CURVE
DENSITY #/C.F.
114.0
• — AVERAGE W
x — MAXIMUM W
o — MINIMUM W
112.0
12.0
14.0
16.0
18.0
WATER CONTENT (W) — %



10
8
6
CYCLES
4
2
0
• - AVERAGE W
x - MAXIMUM W
o - MINIMUM W
6.0
CL SOIL CHANGES
MOD. PROCTOR DENSITY SERIES
4.0
STRENGTH KG/SQ.CM.
2.0
NOTE -: TRANSFERRED DIRECTLY FROM PLATE 12
0.0
125.0
MOD. PROCTOR DENSITY CURVE
100 % S
DENSITY #/C.F.
123.0
• - AVERAGE W
x - MAXIMUM W
o - MINIMUM W
121.0
9.0
11.0
13.0
15.0
WATER CONTENT (W) - %


10
8
6
CYCLES
4
2
0
• – AVERAGE W
x – MAXIMUM W
o – MINIMUM W
3.0
C I (1) SOIL CHANGES
STD. PROCTOR DENSITY SERIES
2.0
STRENGTH
KG/SQ.CM.
1.0
NOTE –: TRANSFERRED DIRECTLY FROM PLATE 13
0.0
100 % S
110.0
STD. PROCTOR DENSITY CURVE
108.0
DENSITY
#/C.F.
106.0
• – AVERAGE W
x – MAXIMUM W
o – MINIMUM W
104.0
15.0
17.0
19.0
21.0
WATER CONTENT (W) – %


10
8
6
CYCLES
4
2
0
• – AVERAGE W
x – MAXIMUM W
o – MINIMUM W
14.0
CI (1) SOIL CHANGES
MOD. PROCTOR DENSITY SERIES
10.0
STRENGTH
KG/SQ.CM.
6.0
NOTE – TRANSFERRED DIRECTLY FROM PLATE 13
2.0
125.0
• – AVERAGE W
x – MAXIMUM W
o – MINIMUM W
100 % S
MOD. PROCTOR
DENSITY CURVE
123.0
DENSITY
#/C.F.
121.0
119.0
9.0
11.0
13.0
15.0
WATER CONTENT (W) ~ %

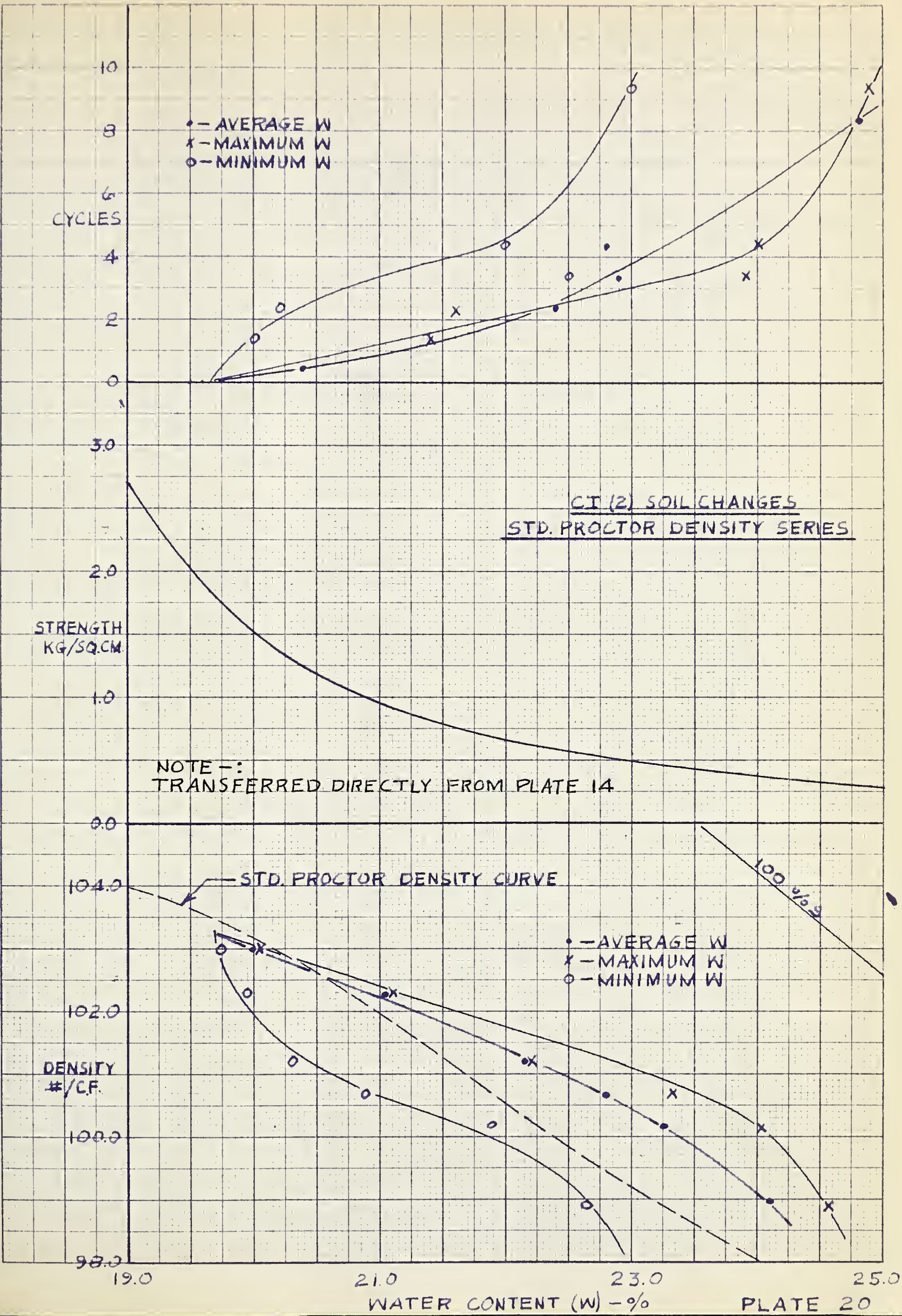
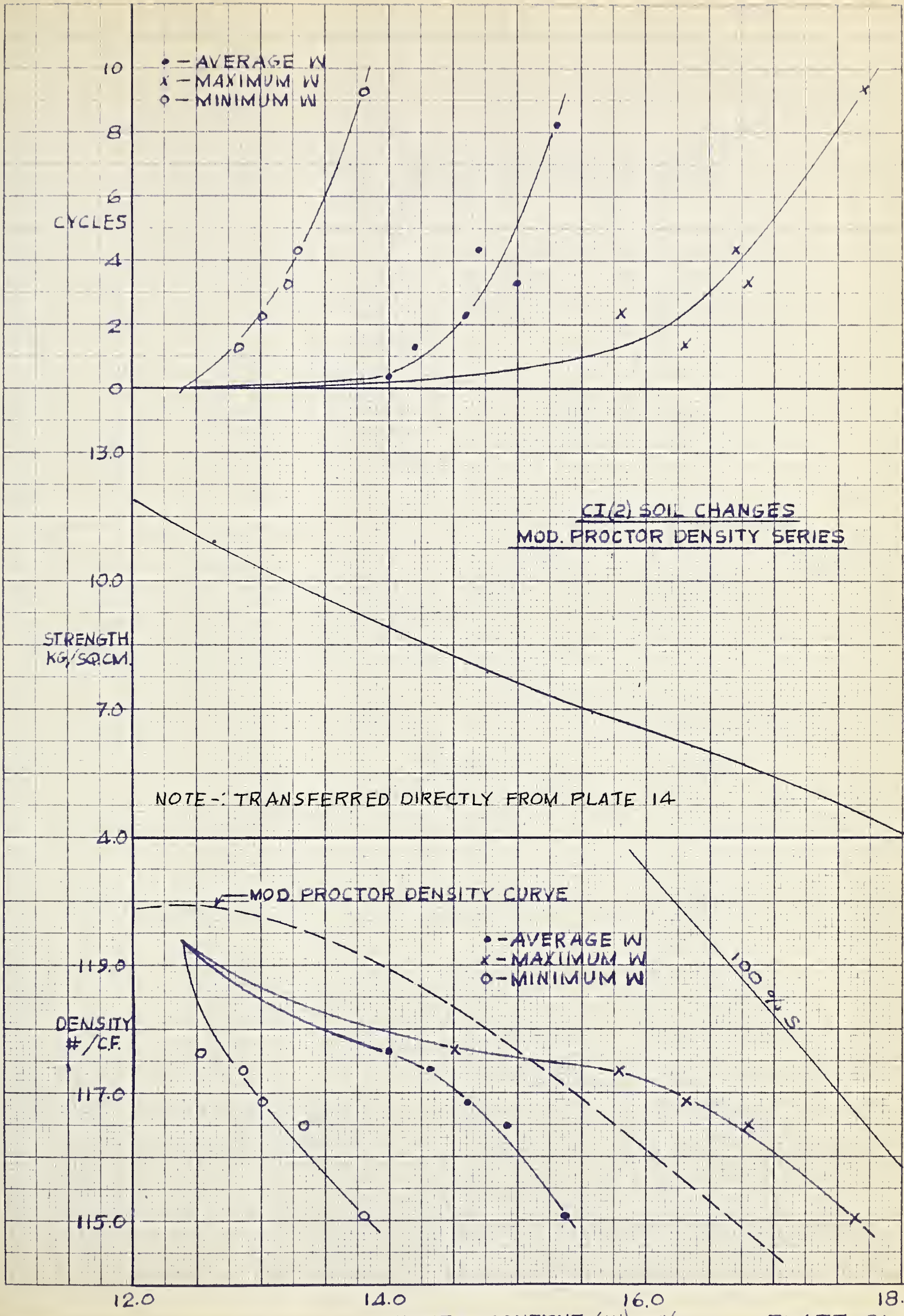
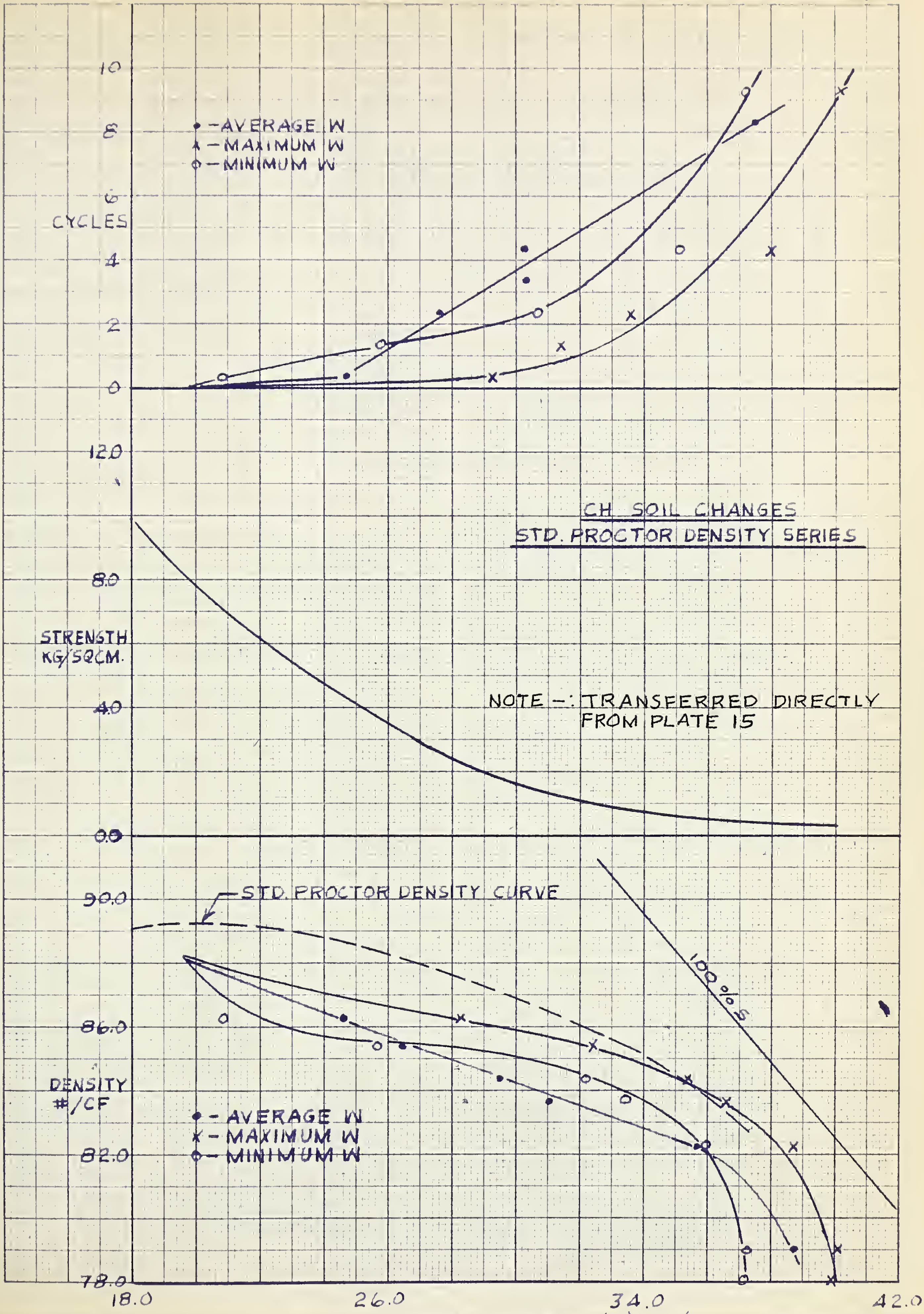

10
8
6
4
2
0
CYCLES
• — AVERAGE W
x — MAXIMUM W
o — MINIMUM W
CI (2) SOIL CHANGES
STD. PROCTOR DENSITY SERIES
3.0
2.0
1.0
0.0
STRENGTH KG/SQ.CM.
NOTE —:
TRANSFERRED DIRECTLY FROM PLATE 14
STD. PROCTOR DENSITY CURVE
100 % S
104.0
102.0
100.0
98.0
DENSITY #/C.F.
• — AVERAGE W
x — MAXIMUM W
o — MINIMUM W
19.0
21.0
23.0
25.0
WATER CONTENT (W) — %

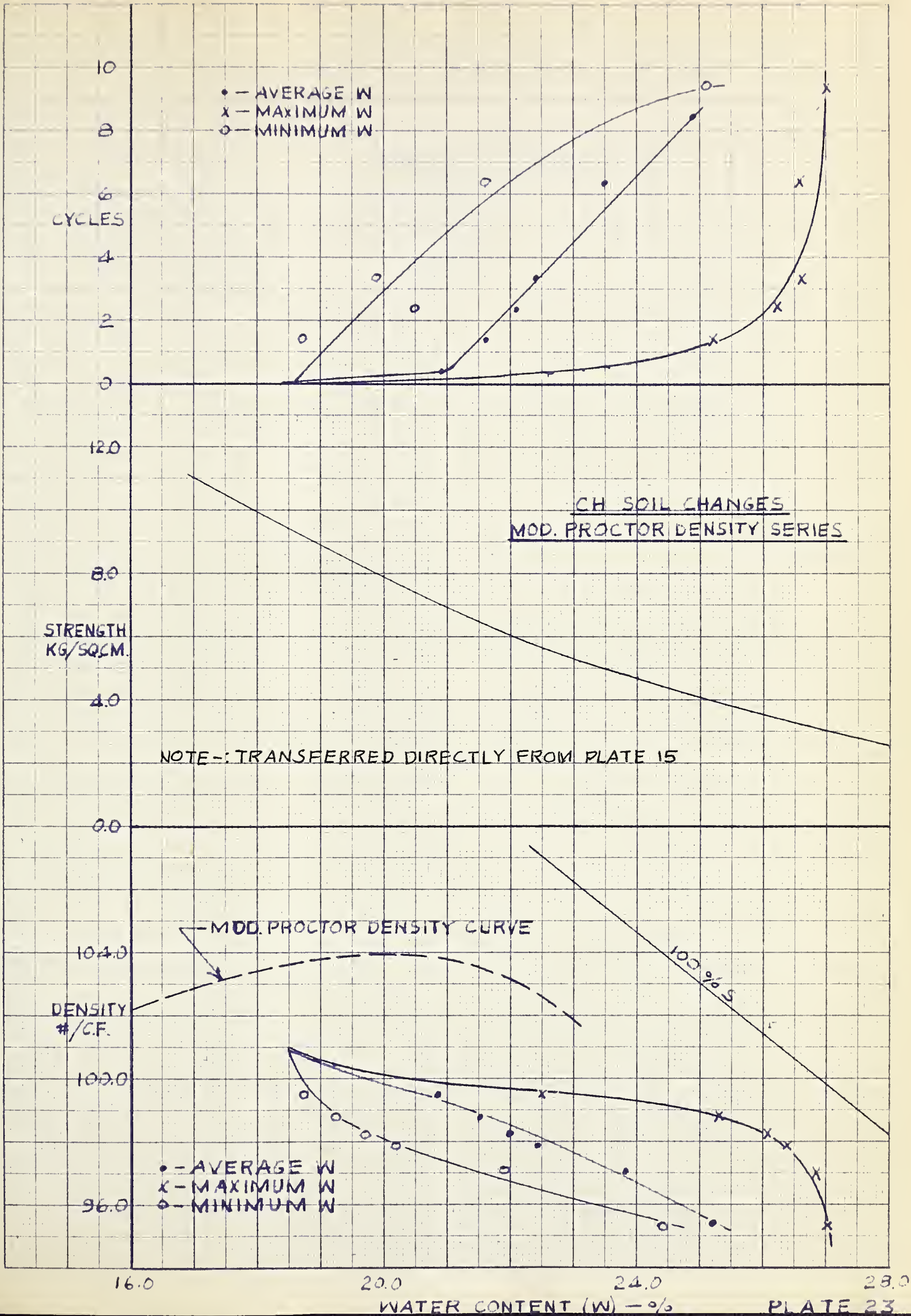
PLATE 20

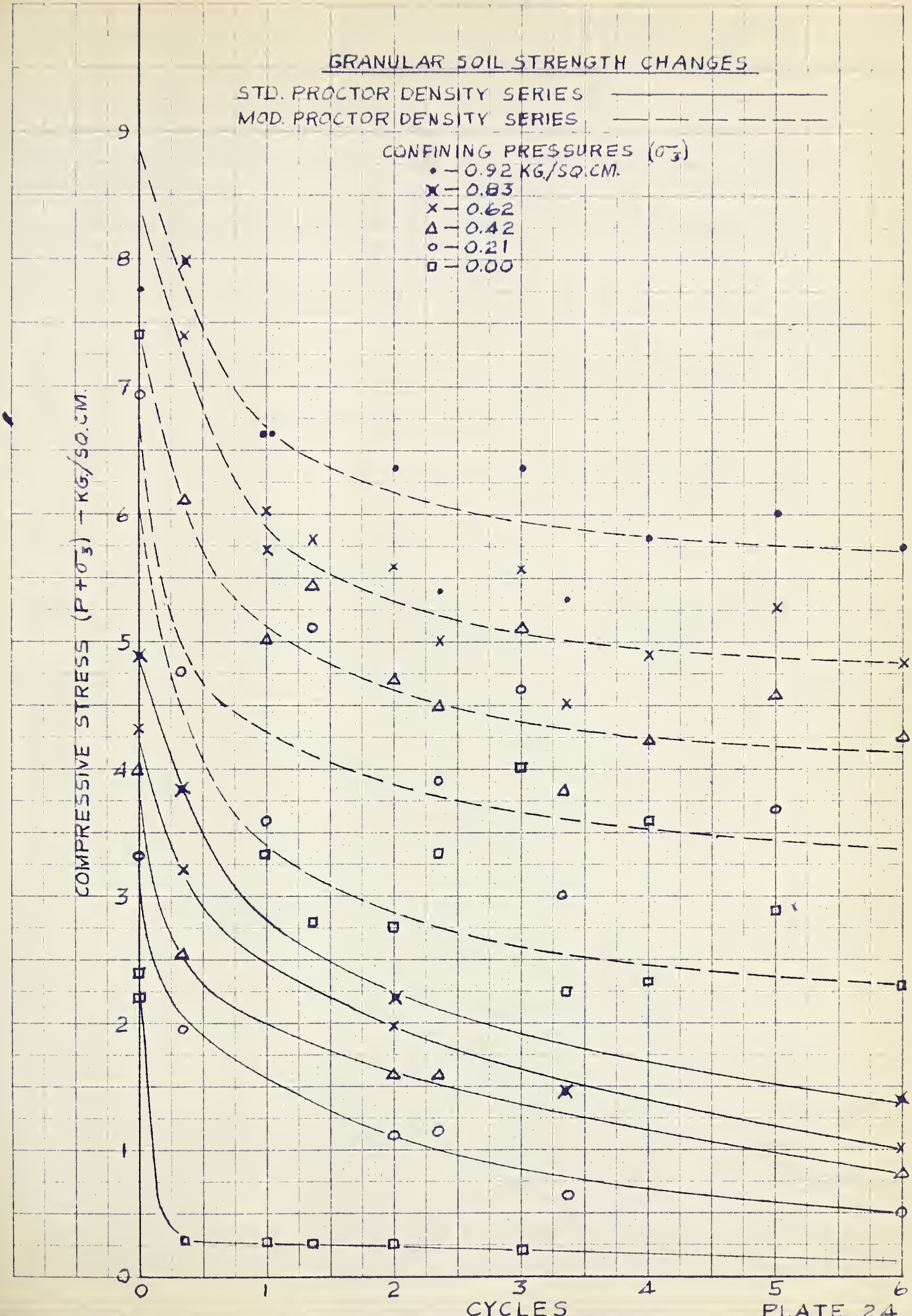
 

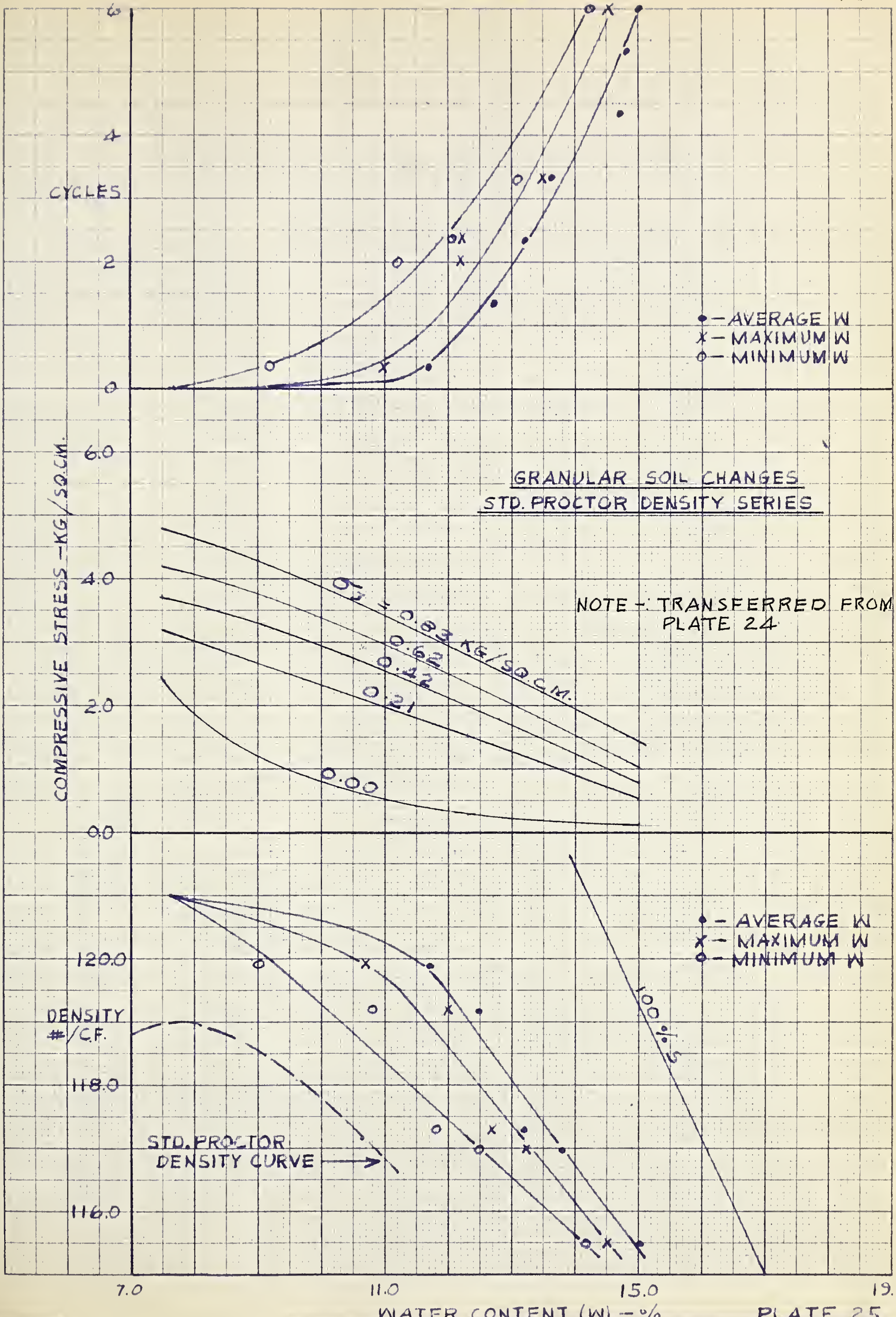


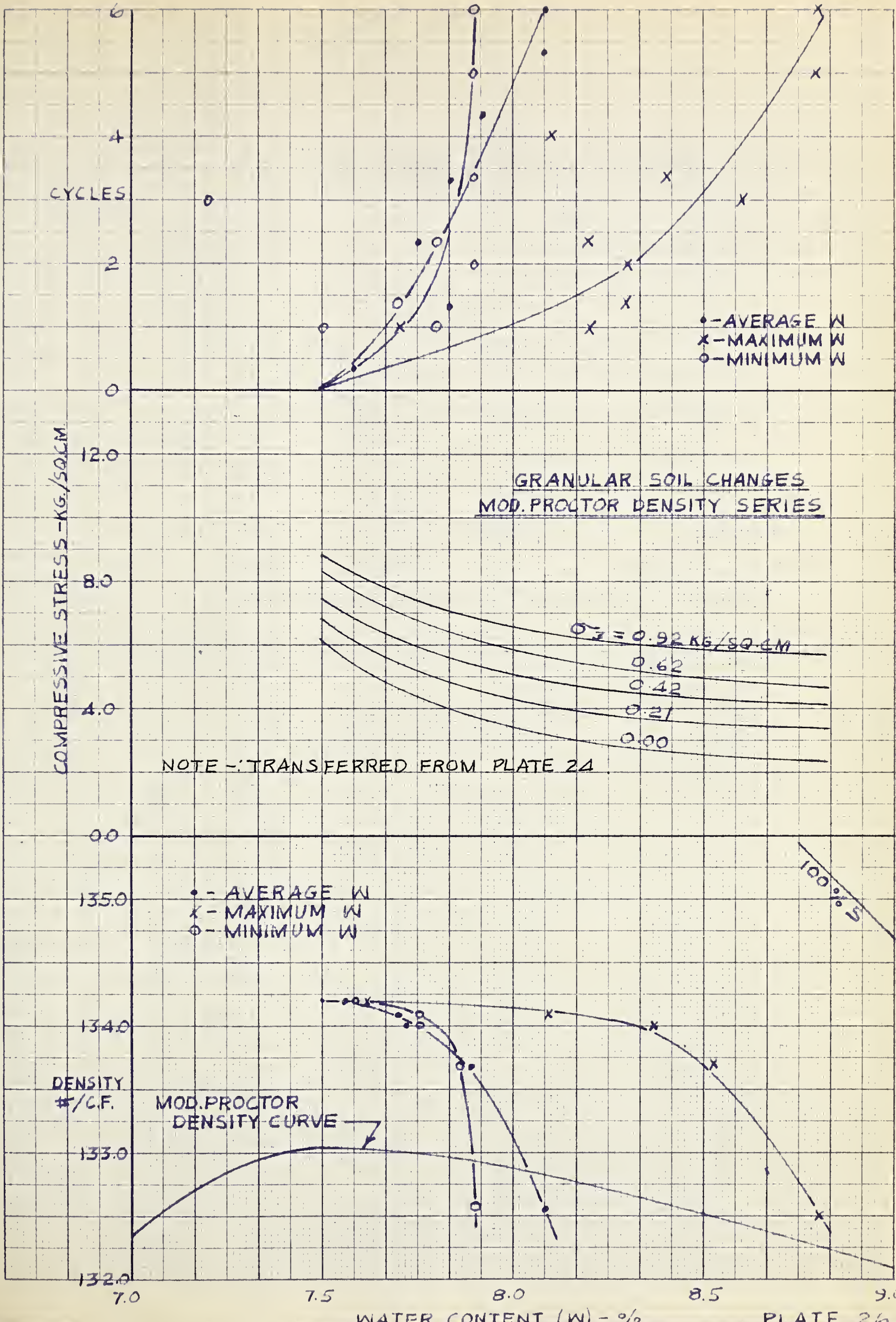
• — AVERAGE W
x — MAXIMUM W
o — MINIMUM W
10
8
6
CYCLES
4
2
0
13.0
CI(2) SOIL CHANGES
MOD. PROCTOR DENSITY SERIES
10.0
STRENGTH KG/SQ.CM.
7.0
NOTE -: TRANSFERRED DIRECTLY FROM PLATE 14
4.0
MOD. PROCTOR DENSITY CURVE
• -AVERAGE W
x -MAXIMUM W
o -MINIMUM W
100 % S
119.0
DENSITY #/C.F.
117.0
115.0
12.0
14.0
16.0
18.0
WATER CONTENT (W) - %

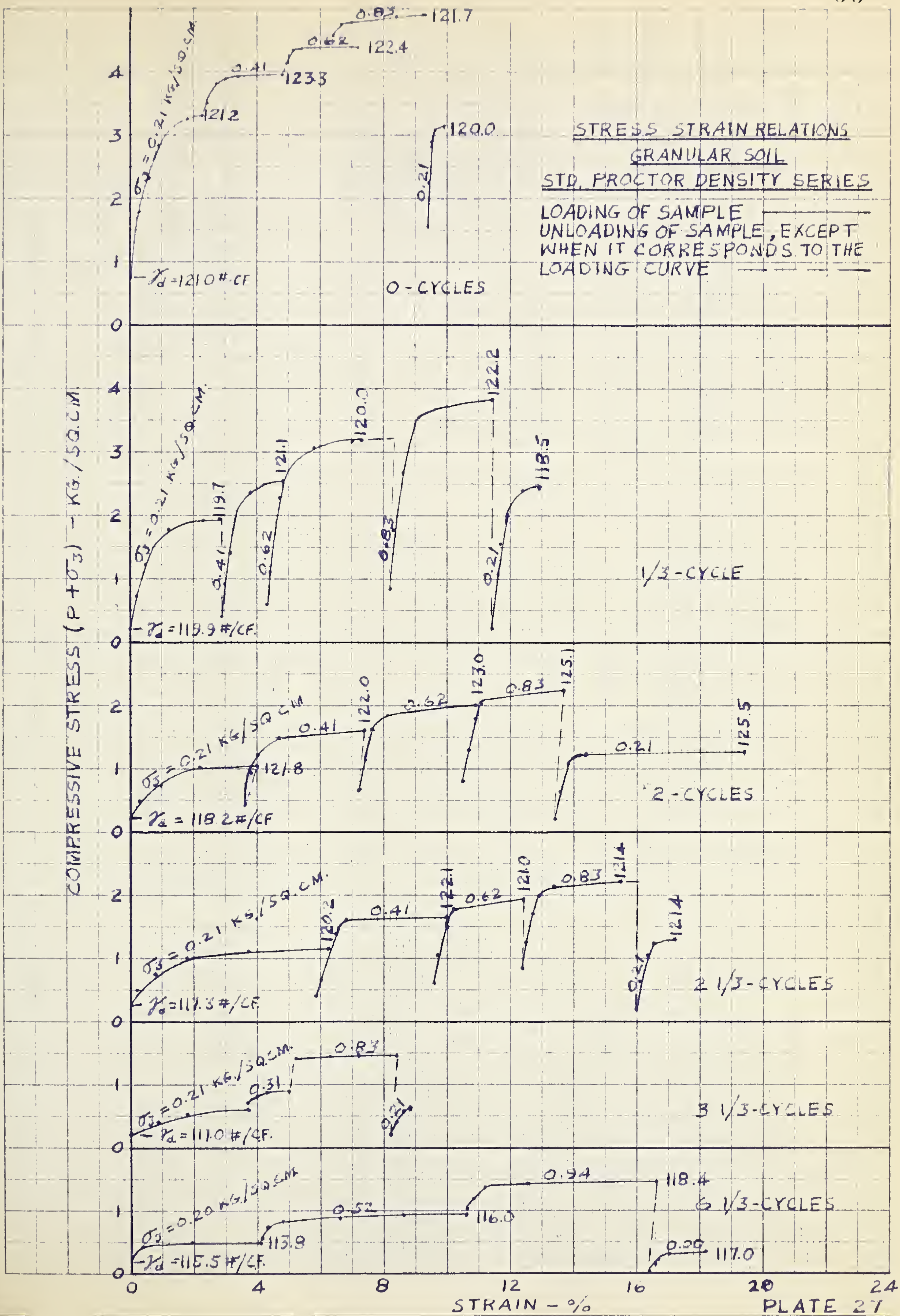

10
8
6
CYCLES
4
2
0
• - AVERAGE W
x - MAXIMUM W
o - MINIMUM W
12.0
8.0
STRENGTH KG/SQ.CM.
4.0
0.0
CH SOIL CHANGES
STD. PROCTOR DENSITY SERIES
NOTE -: TRANSFERRED DIRECTLY FROM PLATE 15
90.0
STD. PROCTOR DENSITY CURVE
100 % S
86.0
DENSITY #/CF
• - AVERAGE W
x - MAXIMUM W
o - MINIMUM W
82.0
78.0
18.0
26.0
34.0
42.0
WATER CONTENT (W) - %

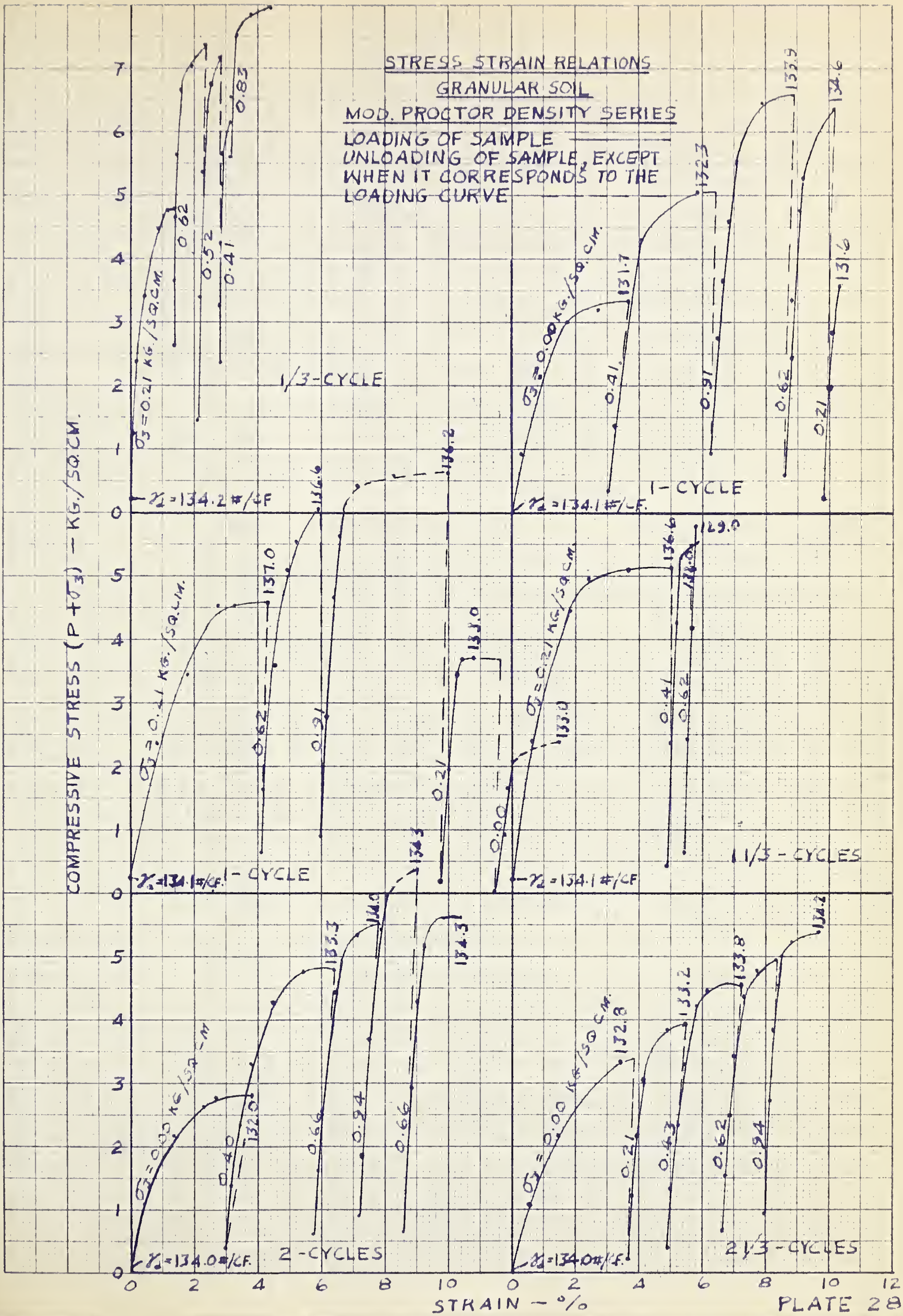

10
8
6
CYCLES
4
2
0
• — AVERAGE W
x — MAXIMUM W
o — MINIMUM W
12.0
CH SOIL CHANGES
MOD. PROCTOR DENSITY SERIES
8.0
STRENGTH KG/SQ.CM.
4.0
NOTE -: TRANSFERRED DIRECTLY FROM PLATE 15
0.0
MOD. PROCTOR DENSITY CURVE
104.0
100 % S
DENSITY #/C.F.
100.0
• - AVERAGE W
x - MAXIMUM W
o - MINIMUM W
96.0
16.0
20.0
24.0
28.0
WATER CONTENT (W) — %

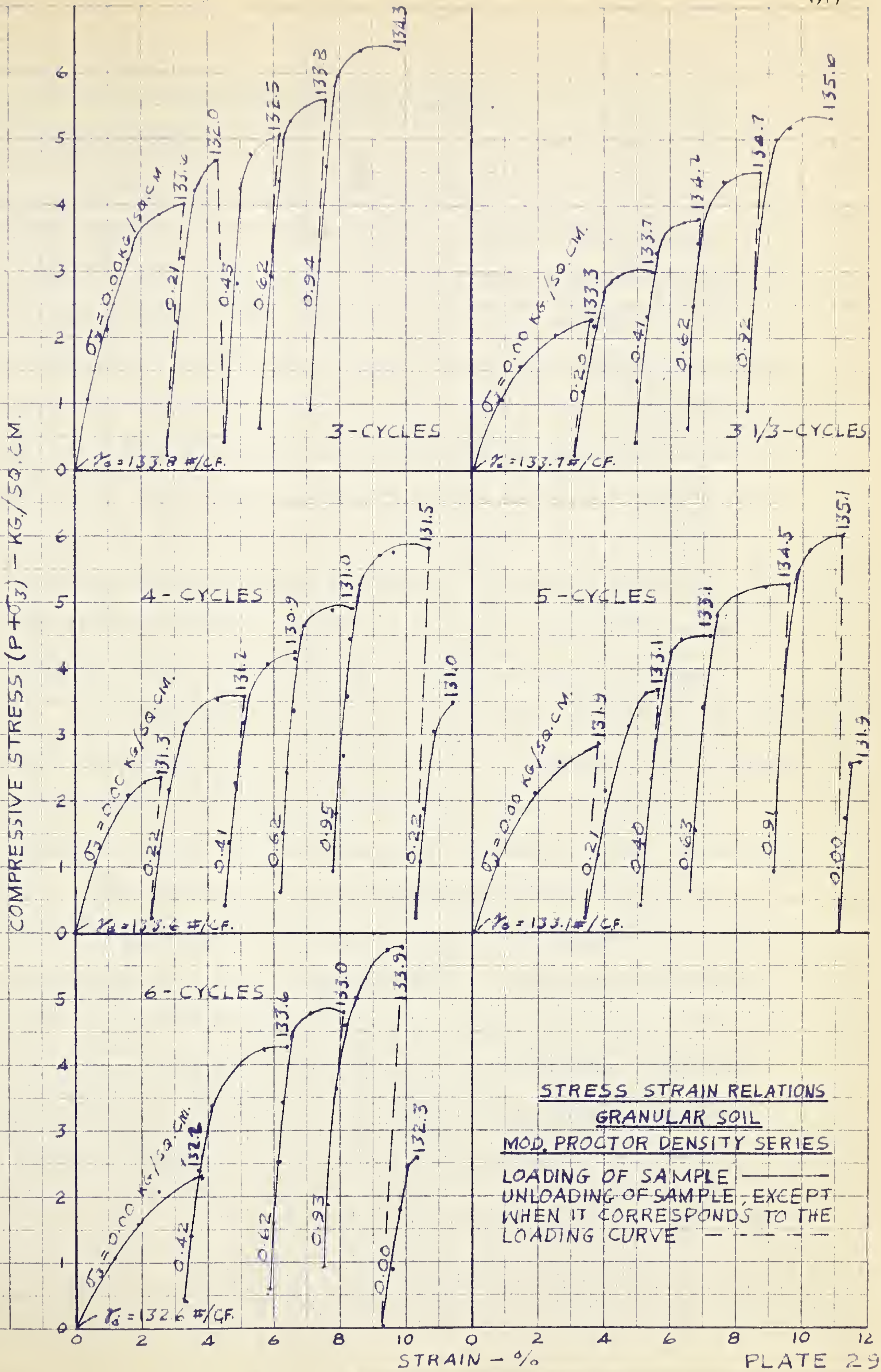

GRANULAR SOIL STRENGTH CHANGES
STD. PROCTOR DENSITY SERIES
MOD. PROCTOR DENSITY SERIES
CONFINING PRESSURES ($\sigma_3$)
• — 0.92 KG./SQ.CM.
— 0.83
× — 0.62
Δ — 0.42
○ — 0.21
□ — 0.00
COMPRESSIVE STRESS ($P+\sigma_3$) — KG./SQ.CM.
CYCLES
0
1
2
3
4
5
6
7
8
9


CYCLES
0
2
4
6
• — AVERAGE W
x — MAXIMUM W
o — MINIMUM W
GRANULAR SOIL CHANGES
STD. PROCTOR DENSITY SERIES
COMPRESSIVE STRESS - KG/SQ.CM.
0.0
2.0
4.0
6.0
σ3 = 0.83 KG/SQ.CM.
0.62
0.42
0.21
0.00
NOTE - TRANSFERRED FROM PLATE 24
DENSITY #/C.F.
116.0
118.0
120.0
100% S
STD. PROCTOR DENSITY CURVE
• — AVERAGE W
x — MAXIMUM W
o — MINIMUM W
7.0
11.0
15.0
19.0
WATER CONTENT (W) - %


CYCLES
COMPRESSIVE STRESS - KG./SQ.CM.
DENSITY #/C.F.
• - AVERAGE W
X - MAXIMUM W
O - MINIMUM W
GRANULAR SOIL CHANGES
MOD. PROCTOR DENSITY SERIES
σ3 = 0.92 KG/SQ.CM
0.62
0.42
0.21
0.00
NOTE -: TRANSFERRED FROM PLATE 24
100% S
MOD. PROCTOR DENSITY CURVE
WATER CONTENT (W) - %


STRESS STRAIN RELATIONS
GRANULAR SOIL
STD. PROCTOR DENSITY SERIES
LOADING OF SAMPLE
UNLOADING OF SAMPLE, EXCEPT WHEN IT CORRESPONDS TO THE LOADING CURVE
0 - CYCLES
1/3 - CYCLE
2 - CYCLES
2 1/3 - CYCLES
3 1/3 - CYCLES
6 1/3 - CYCLES
COMPRESSIVE STRESS (P + σ3) - KG./SQ.CM.
STRAIN - %


STRESS STRAIN RELATIONS
GRANULAR SOIL
MOD. PROCTOR DENSITY SERIES
LOADING OF SAMPLE
UNLOADING OF SAMPLE, EXCEPT WHEN IT CORRESPONDS TO THE LOADING CURVE
1/3-CYCLE
1-CYCLE
1-CYCLE
1 1/3-CYCLES
2-CYCLES
2 1/3-CYCLES
COMPRESSIVE STRESS (P + σ3) — KG./SQ.CM.
STRAIN — %


COMPRESSIVE STRESS $(P+\sigma_3)$ – KG./SQ.CM.
STRAIN – %
3-CYCLES
3 1/3-CYCLES
4-CYCLES
5-CYCLES
6-CYCLES
STRESS STRAIN RELATIONS
GRANULAR SOIL
MOD. PROCTOR DENSITY SERIES
LOADING OF SAMPLE
UNLOADING OF SAMPLE, EXCEPT WHEN IT CORRESPONDS TO THE LOADING CURVE

## Chapter V - ANALYSIS OF TEST RESULTS

### Density Changes

With the exception of the CL soil, the soils compacted to the standard Proctor density exhibited a greater percentage loss in density than those compacted to the modified Proctor density for the number of cycles which were carried out (Plates 8 and 30). The discrepancy between the CL soil and the other soils may be attributed to an increase in density, which was in the order of 0.2 percent, during the first two cycles for the CL soil compacted to standard Proctor density. This increase in density occurred during saturation by capillarity and may be explained on the basis that the capillary forces were not fully developed at the optimum water content.

It may be noted from Plate 30 that the greater percentage loss in density was associated with the soils of the higher plasticity. The maximum percentage loss (10.4%) was found in the CH soil compacted to standard Proctor density.

From the slopes of the average density vs cycle curves for the soils (Plate 8), it would appear that the greater decreases in density of the soils were associated with those soils with the lower initial densities. The rate of change in density was found to be relatively constant and could be expected to remain the same for a maximum number of cycles.

From Plate 8 it may be noted that several of the soils (particularly the CH soil) increased in density during freezing over the first few cycles. With an increased number of cycles, which corresponds to an increase in water content, these soils decreased in density due to freezing. Thus, it may be concluded that there is a critical water content at which neither an increase nor decrease in density occurs due to freezing. For those soils which did not exhibit an increase in density during the initial stages of freezing, it may be assumed that the water content on compaction was higher than the critical water content. The variation of water content throughout the sample made it impossible to determine the critical conditions.

The decrease in density on freezing may be explained, according to Campen and Smith[7] , by the high co-efficients of contraction of the soil water which exists as thin layers. Another explanation, proposed by Beskow[2] , is that when segregated ice is formed from water in the soil, the withdrawal of water from one point to form ice at another may at first result in shrinkage. Since ice segregation was kept to a minimum in the test procedure in this investigation, the increase in density of the samples was probably due to the high co-efficient of contraction of the soil water which existed as thin layers.

An investigation was carried out by Lang[6] under a similar testing procedure to that which was used in this program, except that the soils were frozen in an open system. His investigation was conducted on four different soils, compacted at different water contents. It was found that there was no appreciable decrease in density after three cycles of freezing, thawing and saturation by capillarity. For a clay soil which was tested, the density was found to decrease approximately 20 percent. This test procedure may be considered extremely severe, and not likely to be reproduced in the field. By comparing the loss of density which was reported by Lang to that which was determined in this investigation, it may be concluded that a decrease in density of a compacted soil in a closed system is relatively small as compared to an open system.

Strength Changes

On the basis of the subgrade soil data, it would appear that the most significant direct comparison of data could be made by the relationship of unconfined compression strength vs water content at the end of the test (Plates 12 to 15). The results of the CL and CI (1) soils (Plates 12 and 13), indicated that the strength vs water content could be represented by a straight line on a semi-logarithmic plot within the strength ranges encountered. However, the strength and water content results for the CI (2) and CH

soil (Plates 14 and 15) indicated a curved relationship. In obtaining the samples for the unconfined compression test, it was found that the recovery ratio varied from 90 to 100 percent for each of the soils. Thus, the variation of the actual strength values from the assumed strength vs water content relationship may be due to initial differences between the samples.

The analysis of the strengths of the soils on the basis of the water content at failure were in agreement with the conclusion arrived at by Rutledge[16] that for partially saturated cohesive soils, the compressive strength vs water content may be represented by a single curve for the same lateral pressure. From that investigation, it was found that the plot of water content vs strength for a compacted silty clay could be represented by two straight line relations: one line from the lowest strength values which were obtained to 10 Kg. per sq. cm.; the second line from 10 Kg. per sq. cm. to the highest strength values which were obtained. Since a straight line relationship was found for the CL and CI (1) soils in this investigation and not for the CI (2) and CH soils, it may be that the straight line relationship is confined to soils with a lower plasticity than the CI (2) soil.

The average void ratio and the average degree of saturation were plotted against the average unconfined

compression strength at failure (Plates 10 and 11 respectively) to determine whether or not a relationship existed between these variables. For each soil, the average void ratio and the average degree of saturation were determined on the basis of the volume of the sample as it existed in the mold (Appendix C), rather than on the unconfined compression test specimens.

From the curves of the average void ratio vs average strength (Plate 10) it may be seen that the standard and modified Proctor density series curves tend to be parallel, although they are offset from each other. The curve for the CL soil shows that two void ratios could exist for the same strength. In general, the shapes of the curves would indicate that the same condition exists for all the soils. This may be explained on the basis of the water content in the specimen. That is, the soils compacted to the modified Proctor density would exhibit a higher degree of saturation, and would thus possess a lower strength at the same void ratio.

From Plate 10, it would appear that the larger changes in the average void ratio were associated with those soils with the higher swelling characteristics. This would indicate that the decreases in the density of the soils may be primarily attributed to swelling. However, the freezing process may change the soil structure so that additional water may enter the soil, thus providing water for the swelling to occur.

The increase in the void ratio of the CL soil, which could be expected to exhibit low swelling characteristics, was found to be in the order of 0.02. The void ratio for the CH soil, which would exhibit high swelling characteristics, was found to increase from 0.99 to 1.33 for the standard Proctor density series. The smaller increase (0.12) in the void ratio of the CH soil compacted to modified Proctor density may be due to the lower permeability, which would permit less water to be taken up into the soil for the swelling process.

The average degree of saturation vs average strength (Plate 11) shows that the curves for the standard and modified Proctor density are displaced from each other. In each case the modified Proctor density series curves indicates higher average strength values for the same average degree of saturation. This may be explained on the basis of the higher void ratio which existed for the standard Proctor density at the same degree of saturation.

In the cases of the CL and the CI (2) soils, the average degree of saturation appears to be approaching a constant value (Plate 11). This may be substantiated by the tendency for the average density curves to become parallel to the 100 percent saturation curves (Plates 16, 17, 20 and 21). This would indicate that the increase in the voids of the soil, due to the freezing and swelling processes, was becoming proportional to the quantity of water being taken up into the soil by the capillary forces.

Since the soils, other than the CL and CI (2), do not appear to be approaching a constant value of the average degree of saturation up to nine cycles (Plate 11), it could be expected that they would appraoch a constant value with an increased number of cycles. This was indicated by the tendency of the average density curves to become parallel to the 100 percent saturation curves (Plates 16 to 26).

As compared to the plots of average void ratio and average degree of saturation vs average strength, the plot of the average water content vs average strength (Plate 9) indicated that these two variables tended to follow the same curve for each of the soils, for both standard and modified Proctor density series. This plot shows that the larger changes in the average water content were associated with those soils with the higher plasticity.

The plots of the average void ratio, average degree of saturation, and average water content vs average strength, would indicate misleading strength values due to the averaging of the variables. In each case, the variable plotted against the average strength was based on the total sample as it existed in the mold, as compared to particular portions of the sample which were taken for the strength tests, and on which water content determinations were carried out. Thus, it was decided to employ the relationship of water content vs strength, (Plates 12 to 15), for the analysis of the strength changes, since this would include the immediate effect of a

change in water content on the strength of the soil. In addition, the plots of water content vs strength were substantiated by a far greater number of test points than were the other plots.

The minimum strength existing at each stage of cycling was determined from the maximum water content found in the specimen. From an examination of the strength and cycles vs water content curves (Plates 16 to 26), it was found that the rate of decrease in the minimum unconfined compression strength was markedly reduced after the first two cycles for all the soils.

The location of the minimum and maximum water contents found in the soil specimen depended on the type of soil, the density and the stage of testing (Appendix A and B). The maximum water content was located at the bottom of the sample for all the soils compacted to modified Proctor density, with the exception of the CL soil. However, the maximum water content found in the soils compacted to the standard Proctor density followed an erratic pattern. This indicated that there was considerable migration of water, due to freezing, in the soils compacted to the standard Proctor density.

The minimum percentage of the initial strengths which were retained after 9 1/3 cycles for the subgrade soils and six cycles for the granular soil were determined from Plates 16 to 26. These values (Plate 30) were based on the minimum strength which was found at the end of the cycling

period. The percentage of initial strengths which were retained for the granular soil depended on the confining pressure. However, an estimate of the decrease in strength was made on the basis of the average confining pressure.

In each case, except for the CH soil (standard Proctor density series) and the CL soil (modified Proctor density series), the percentages of the initial strength as determined from the strength vs water content criteria (Plates 12 to 15) were in close agreement with the strength losses as checked from the results of the strength tests (Appendix A and B) (Plate 30). The discrepancy of the CH soil (standard Proctor density series) and CL soil (modified Proctor density series) with the strength criteria may be due to the position of the assumed strength vs water content curves (Plates 12 and 15), or to misleading unconfined compression strength values. Since each of the strength losses as determined from the strength vs water content criteria and from the results of the strength tests were in agreement for other than these two soil series, it would appear that the unconfined compression strength values were in error. This may be substantiated by the difficulty in obtaining undisturbed samples for the CL soil (modified Proctor density series) and the CH soil (standard Proctor

density series). Thus it may be concluded that the strength losses as obtained from the strength vs water content criteria are more reliable than those as determined from the results of the strength tests.

For the soils of the standard Proctor density series, the percentage of the initial strength decreased as the plasticity of the soils increased (Plate 30). The CL soil maintained 44 percent of its initial strength as compared to four percent for the CH soil. The percentage of initial strength for the modified Proctor density series varied from approximately 55 percent for the granular soil to 22 percent for the CI (1) soil.

The minimum strength ratio was employed to determine the ratio of the unconfined compression strength of the subgrade soils compacted to modified and standard Proctor density at the same number of cycles (Plate 30). The minimum strength was obtained from the strength corresponding to the maximum water content from Plates 16 to 23.

From Plates 16 and 17 it was found that the minimum strength ratio for the CL soil in compaction was 2.8 as compared to 1.7 after nine cycles (Plate 30). In determining the minimum strength for the modified Proctor

series, the maximum water content curve was employed, although the average water content curve indicated higher water contents. The maximum water content curve was adhered to because the maximum water contents were obtained from water content determinations throughout the sample, whereas the average water content was calculated from the amount of water in the soil as it existed in the mold.

By comparing the cycles vs water content plots on Plates 16 and 17, it was found that the modified Proctor density series of the CL soil was increasing in water content at a higher rate at the end of nine cycles than was the standard Proctor density series. This would indicate that the minimum strength ratio would decrease below 1.7 with an increased number of cycles since the strength was assumed to be a function of the water content which depends on the number of cycles. Thus, the higher initial density would not be warranted for this soil.

The minimum strength ratios for the CI (1) and the CI (2) soil were approximately constant for zero and nine cycles (Plate 30). From the plots of cycles vs water content (Plates 18 to 21), it may be seen that for the CI (1) and CI (2) soil, the standard Proctor density series was

increasing in water content at a higher rate, after nine cycles, than was the modified Proctor density series. Since the strength was assumed to be a function of water content, the minimum strength ratio would increase with an increased number of cycles. Thus, the CI (1) and the CI (2) soils should be compacted to the maximum possible density.

Although the CH soil exhibited a very low initial minimum strength ratio (1.2), at nine cycles it increased to 10.0. By comparing the plots of cycles vs water content (Plates 22 and 23) it may be concluded that this value would become even higher with an increased number of cycles, due to the higher rate at which the standard Proctor density series was increasing in water content at the end of nine cycles. From this, it may be concluded that this soil should be compacted to the maximum possible density.

From the results of the granular material (Plates 24, 25 and 26) the decrease in strength appears to be negligible beyond six cycles for the soil compacted to the modified Proctor density. The soil compacted to the standard Proctor density appears to decrease in strength beyond six cycles. Although the strength ratio depends on the confining pressure, it was estimated on the basis of the average confining pressure to be in the order of five after six cycles as compared to two for the soils as

compacted. Thus, the granular soil should be compacted to the maximum possible density.

The differences between the changes in strength of the soils at different densities in this investigation may be explained on the same basis as Linell and Kaplar[26] explain the behaviour of variation in density on the rate of heaving. In silt samples, the effect of lower permeability at the higher unit weights is outweighed by the following factors: an increase in the force of moisture attraction; and possibly the closer packing of the soil grains provides better continuity of the soil pores, with the result that more water can flow into the soil. In clay and well graded soils, the higher unit weights presumably reduce the permeability so that this advantage outweighs the previously mentioned factors, which probably have their maximum effectiveness at the lowest degree of density.

The decrease in strength for each of the soils may be attributed to the increase in the water content. From Plates 16 to 26 it was found that the greatest decrease in strength occurred during the first few cycles in which the changes in density were negligible. Since the initial density influences the changes in water content which occur due to saturation by capillarity, it should be considered as the most important factor, with regard to the changes in strength in this investigation.

According to Turner[25] ".... for any soil there is a definite relationship between moisture content and bearing capacity, the higher bearing capacities being associated with low moisture contents". He found that the spring loss of subgrade bearing strength was in the order of 50 percent, as determined by the CBR test, with variations of water content from 5 to 10 percent depending on the soil type and conditions. Although similar losses of strength have been reported by Meskal[12] and Davis[13] , these investigators found that the small variations in water content were insufficient to account for all the loss of strength. Meskal suggested that the stability of the soil mass was affected by altering the soil structure without necessarily changing the water content during freezing. Seed and Chan[23] found that thixotropic action* could increase the strength up to 50 percent in some compacted clays after a period of storage, especially in soils with a high degree of saturation. By applying this phenomena to subgrade soils, it is possible that the increases in strength in the fall may be partially due to the thixotropic effect which would be destroyed by the freezing of the soil moisture.

The losses in strength as determined in this investigation were greater than those which are normally encountered in the field. This may be explained on the basis of the time effect. In the field, soils have a

* The increase in the cohesive strength due to the time effect.

period of approximately six months in which excess water may be drained, and during which an increase in strength may be developed due to thixotropic action. However, in this laboratory investigation, any increases in strength which may have developed due to the time effect were neglected due to the cycling procedure which was employed. In addition, the test conditions may be considered to be somethat more extreme than is normally encountered in the field.

From a study of freezing and thawing tests on undisturbed clays, Curtis and Dickson[24] found that the unconfined compressive strength decreased due to freezing. If the soil was assumed to be saturated, a decrease in density would occur on freezing which would tend to destroy the strength which could be attributed to the thixotropic effect. A part of the strength loss was regained with time, and the amount of regain depended on the soil type. Thus, from the results of the tests performed by Curtis and Dickson[24], as well as Seed and Chan[23], it may be reasoned that the time effect may have considerable influence on the decreases in strength in this investigation.

Testing Procedure

The testing procedure developed for this program may be considered to be much more severe than the conditions existing in the field, with the possible exception of ice

lens formation. The severity of this testing procedure may be attributed to the following: the time effect*, the availability of water, and, the effect of a surcharge.

In the field, the availability of the water to the soil at a particular depth between the road surface and the water table depends on the position of the water table. Since the water table varies throughout the year, the availability of water to the soil at a particular depth would be less severe than the condition existing in this laboratory investigation, in which the water was maintained at the bottom of the sample, during the period in which the sample was thawed.

In the field, soils are subjected to a surcharge, the magnitude of which depends on the position below the road surface. Since the effect of a surcharge is critical in the prevention of heave for a soil in which ice lensing occurs[5], it would be insignificant in a saturation soil in which the expansion of the water on freezing provided the expansion forces in the soil. Beskow[2] found that when a partially saturated sand sample was frozen in one direction, the heave was several times greater than that which could be attributed to the volume change of the contained water on freezing, and that the expansion was affected by the surface loading. Since the soils in this investigation were

* See pages 82 and 83 for a discussion of the time effect.

partially saturated and subjected to a quick freezing condition from all directions, in which no surcharge was employed, no means was available to estimate the effect of a surcharge on the strength losses

An attempt was made to determine the water content variation after thawing by obtaining the water content variation in the samplers which were sealed. Since the sealed samples were not tested until the remainder of the soil had become capillary saturated, the tendency was for the water content to become uniform throughout the sealed sample. Thus, an accurate distribution of moisture on thawing could not be obtained by this method. However, the procedure was continued throughout the investigation to substantiate the strength vs water content curves, as well as to check on any increase in strength due to the time effect during the period in which the samplers were sealed. From the strength vs water content curves, no increase in strength was apparent due to the time effect.

The average water content as calculated from the weight of the water in the sample, as it existed in the mold (Appendix C) tended to be less than the average water content as determined by carrying out the actual water content measurements on the sample (Appendix A). In all cases, except for the CH soil, the variation was less than one percent which would be relatively insignificant compared to

the water content changes which occurred. The average water contents based on the increase in the weight of water in the samples, as they existed in the mold, should be considered as the most accurate since they were taken as the average of the samples at each stage of testing, and the samples were not subjected to the consolidating effect of the sampling tubes being driven into the soil.

The multi-stage vacuum triaxial test appeared to be successful in determining the changes in the strength of the granular soil. The stress strain relationships for this soild were presented on Plates 27, 28 and 29. It may be noted that in each stage of testing the peak stress was very closely approximated by the criterion which was employed for failure conditions. From the sahpe of the stress strain curves, an increase in density of the material would be indicated due to the steepening of the stress strain curve after the first stage of testing. It would appear that after the first stage of testing, there was little change in the structure of the sample, since the stress strain curves were approximately parallel.

An attempt was made to determine the density of the soil at the end of each stage of testing. This was based on the volume of the sample as obtained from the average of the middle and end areas multiplied by the heighth of the sample. Thus, the volume due to the curvature of the

specimen was neglected, which would tend to give slightly lower values of density than are shown on Plates 27, 28 and 29. From these plates, it may be seen that there is a tendency for the specimens to decrease in density during the first stage of testing, after which an increase in density occurred with an increase in the confining pressure which appears to be in agreement with that indicated by the slope of the stress-strain curves. In general, the maximum change in density which occurred was in the order of two percent, based on the approximated densities. However, the error introduced due to the curvature of the specimen was not known, but could be expected to be larger than the errors introduced due to the measurements.

Although the strength values obtained from this test appear reasonable, no definite conclusions as to the validity of the strength values could be drawn without additional testing to determine what was happening to the structure of the soil. Since the height to diameter ratio was 1.2, the strength values obtained would probably be too high. However, no correction was applied for the length to diameter ratio, since only relative values were required.

## CONCLUSIONS

The analysis of the tests performed for this investigation indicated that:

1. The percentage of the initial strengths retained fro the subgrade soils varied from four to 44 percent for the standard Proctor density series, from 0 to 9 1/3 cycles. The higher losses in strength were associated with those soils with the higher plasticity. For the modified Proctor density series, the percentage of the initial strength retained varied from 22 to 42 percent, with an average value in the order of 30 percent. After six cycles, the strength of the granular soil was reduced to approximately 55 percent of its original strength for the modified Proctor density series, and approximately 22 percent for the standard Proctor density series. With the possible exception of ice lens formation, the magnitude of the strength losses in this laboratory investigation could not be expected in the field due to the severity of the testing procedure.

2. The rate of decrease in strength was greatly reduced after the first two cycles for all the soils.

3. The greatest decrease in density (10.4%) occurred in the CH soil compacted to the standard Proctor density. The decrease in the density of the other soils were in the

order of five percent or less. It would appear that the decrease in density was due primarily to the swelling of the soil, and that freezing altered the soil structure, permitting water to be available for the swelling to occur.

4. The decreases in the initial strengths are dependent on the initial compacted density, since this governs the water content changes which occur. For each of the subgrade soils, it was found that the relation between the unconfined compression strength and the water content could be represented by a single curve.

5. With the exception of the CL soil, the soils should be compacted to the highest possible density. The benefits to be expected appear to increase as the plasticity of the soil increases. From the analysis of the CL soil, a density higher than standard Proctor density would not be warranted.

## Recommendations for Future Research

1. The changes in the initial dry density and the strength could be further evaluated by a similar testing procedure which was employed in this investigation, with the exception that the specimens would be frozen from the top in a closed system. In addition, the effect of a surcharge should be studied. These test conditions could be considered to represent the most severe conditions existing in the field, with the exception of freezing in an open system. From the results obtained, a broader comparison could be made as to the merits of the different compaction efforts in a closed system.
2. The results of these studies indicate that the loss of strength is due to the variation in the water content. Since other investigators[12,13] have not found this to be the case in the field conditions, an attempt could be made to investigate the fundamental mechanism occurring during the cycling procedure. This would involve a study of the time effect as well as the conditions existing during the freezing process. The effect of a surcharge should also be investigated. With an analysis of these variables, the strength losses as obtained in this program could be more favourably correlated with the field conditions.

3. The performance of the multi-stage vacuum triaxial test indicated that such a test could be developed and used successfully as a means of evaluating the shear strengths of base course materials.

| SOIL | MIN. STRENGTH RATIO | | CHANGES IN PROCTOR DENSITY | |
|---|---|---|---|---|
| | 0 - CYCLES | 9 - CYCLES | STD. PROCTOR SERIES | MOD. PROCTOR SERIES |
| CL | 2.8 | 1.7 | 1.1% (9 cycles) | 1.5% (9 cycles) |
| CI (1) | 5.5 | 4.9 | 3.0 " | 2.0 " |
| CI (2) | 6.2 | 6.3 | 5.1 " | 3.7 " |
| CH | 1.2 | 10.0 | 10.4 " | 5.4 " |
| Gravel | - | - | 4.5 (6 cycles) | 1.2 (6 cycles) |

| SOIL | % OF INITIAL STRENGTH* | | | |
|---|---|---|---|---|
| | STRENGTH VS WATER CONTENT CRITERIA | | RESULTS OF STRENGTH TESTS | |
| | STD. PROCTOR SERIES | MOD. PROCTOR SERIES | STD. PROCTOR SERIES | MOD. PROCTOR SERIES |
| CL | 44 | 27 | 48 | 40 |
| CI (1) | 25 | 22 | 25 | 27 |
| CI (2) | 17 | 42 | 18 | 45 |
| CH | 4 | 31 | 10 | 32 |
| Gravel | App. 22 | Approx. 55 | Approx. 24 | Approx. 54 |

* After 9 1/3 cycles for subgrade soil and 6 cycles for granular soil.

SUMMARY OF TEST RESULTS

PLATE 30

BIBLIOGRAPHY


1. Taber, S. "Freezing and Thawing of Soils as Factors in the Destruction of Road Pavements". Public Roads, Vol. 11, No. 6 pp. 113-132 August, 1930.

2. Beskow, Gunnar (Ph.D.) "Soil Freezing and Frost Heaving with Special Application to Roads and Railroads". 1935 trans. J.O. Ostenberg, Ph.D., Evanston, Illinois. The Technological Institute, Northwestern University, 1947.

3. Benkelman, A.C. and Olmstead, Frank R. "A New Theory of Frost Heaving". Proceedings, Highway Research Board, Vol. 11, pp. 152-165, 1931.

4. Winn, H.F. and Rutledge, P.C. "Frost Action in Highway Bases and Subgrades". Purdue University, Engineering Bul., Vol. 24, No. 2, May, 1940.

5. Haley, James F. and Kaplar, Chester W. "Cold Room Studies of Frost Action in Soils". Symposium in Frost Heave and Frost Action in Soils, Highway Research Board, Special Report No. 2, Jan. 1952.

6. Lang, F.C. Discussion in "Frost Action in Highway Subgrades and Bases". Proceedings, Purdue Conference on Soil Mechanics and its Applications, Purdue University, Symposium on Frost Action, pp. 457-460. Sept. 2-6, 1940.

7. Campen, W. and Smith, J. "Some Physical Properties of Densified Soils". Proceedings Highway Research Board, Vol. 22 pp. 460-469, 1942.

8. Porter, H.C. "Roadway and Runway Soil Mechanics Data". Proceedings Highway Research Board, Vol. 22 pp. 469-478, 1942.

9. Foster, "Reduction in Soil Strength with Increase in Density" Proceedings, American Society of Civil Engineers, Vol. 120 pp. 803, 1955.

10. Casagrande, A. "Discussion on Frost Heaving". Proceedings Highway Research Board, Vol. 11, pp. 168-172, 1931.

11. Schaible, L. "On Increasing Danger of Frost Damage to our Highway", from Die Boutechnic, Trans. D. Sinclair, National Research Council of Canada, Technical Translation TT-568, 1955.

12. Meskal, G.A. "Final Report of Committee on Load-Carrying Capacity of Roads as Affected by Frost Action" Bulletin 207, Highway Research Board, Jan., 1958.

13. Davis, M.M. "Load Testing Program of Ontario Department of Highways" Proceedings of the Sixth Canadian Soil Mechanics Conference, National Research Council of Canada, pp. 79=93, Dec. 1952.

14. Black, Crosey and Jacobs, "Field Studies of Soil Moisture" Road Research Technical Paper No. 41, 1958, Abstract, Highway Research Board, Feb., 1959.

15. Casagrande, A. "Classification and Identification of Soils", American Society of Civil Engineers, Paper No. 2351, 1948.

16. Waterways Experiment Station, "Triaxial Shear Research and Pressure Distribution in Soils, Vicksburg, Miss., April, 1947.

17. Bishop, A.W. and Henkel, D.J., "The Measurement of Soil Properties in the Triaxial Test", Edward Arnold (Publishers) Ltd., London, 1957.

18. Road Research Laboratory, D.S.I.R. "Soil Mechanics for Road Engineers" Her Majesty's Stationery Office, London, 1957, p. 373.

19. American Society for Testing Materials, "A Triaxial Shear Investigation on a Partially Saturated Soil", Special Technical Publication 106, pp. 180-187, 1951.

20. Lamb, T.W. "Soil Testing for Engineers" The Massachusetts Institute of Technology. John Wiley and Sons, Feb., 1958.

21. American Society for Testing Materials. "Procedures for Testing Soils", 1916 Race St., Philadelphia 3, Pa., April, 1958.

22. Leonards, G. "Strength Characteristics of Compacted Clays". Proceedings, American Society of Civil Engineers, Vol. 120, 1955.

23. Seed, H.B. and Chan, C.K. "Thixotropic Characteristics of Compacted Clays". Journal of the Soil Mechanics and Foundations Division, American Society of Civil Engineers, Vol. 83, No. SM 4, Nov., 1957, Part 1.

24. Curtis, W. and Dickson, W. "The Effect of Freezing and Thawing on the Engineering Properties of Clays and Shales", Unpublished Thesis, Submitted to the School of Graduate Studies in partial fulfilment of the requirements for the degree of Master of Science. Sept., 1955, University of Alberta.

25. Turner, Jr., K.A. "Loss and Recovery of Bearing Capacity of 30 New Jersey Soil Materials as Determined by Field CBR Tests 1954-55. Bulletin 168, Highway Research Board, pp. 9-50 Jan., 1957.

26. Linell, K.A. and Kaplar, C.W. "The Factor of Soil and Material Type in Frost Action", Bulletin 225, Highway Research Board pp. 81-127, Jan., 1959.

# A P P E N D I X   A

## Summary of Subgrade Soil Test Data

The test data for the subgrade soils has been summarized from the test data sheets of which a typical data sheet is shown on the following page. The results of each specimen has been summarized under the following headings:

Cycles - Number of cycles to which the specimen has been subjected.

Density - Average density measured for a given number of cycles.

Strength - Unconfined compression strength.

The asterisk (*) refers to the sealed samples, whereas the other strength values refer to the saturation by capillarity stage of testing (1/3 stages).

Strain - Corresponding to the strength values.

W-top, middle and bottom - Water content found in the top, middle and bottom of the samples.

The average water content (Av.W) has been recorded as the average water content of the soil in the mold after the removal of the unconfined compression test samples. It may be noted that a complete coverage of water contents has been omitted in several of the initial stages of testing. This occurred prior to the standardization of the test procedure.

| UNIVERSITY OF ALBERTA | | MOISTURE DENSITY AND STRENGTH DATA | |
|---|---|---|---|
| UNIT WEIGHT DETERMINATION — MOLD NO. 24 WT., GMS. | 4,567 | DATE | July 3, 1959 |
| WT. SAMPLE WET + MOLD, GMS. | 10,716 | MATERIAL | CI(1) soil |
| WT. SAMPLE WET, GMS. | 6,149 | COMPACTION | Modified Proctor |
| VOLUME MOLD, CF. | 0.1015 | | |
| AVE. MOISTURE CONTENT | 9.3 | | |
| WET UNIT WT., LB/FT$^3$ | 133.3 | | |
| DRY UNIT WT. LB/FT$^3$ | 121.8 | | |

**MOISTURE CONTENT ON COMPACTION**

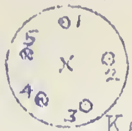
| LAYER | MOISTURE CONTENT |
|---|---|
| 1 | 9.3 |
| 3 | 9.3 |
| 5 | 9.2 |
| AVERAGE | 9.3 |

AVERAGE MOISTURE CONT. AT END = 10.3

% SATURATION AT START = 70%

% SATURATION AT END = 73%

**UNCONFINED COMPRESSION TEST RESULTS**




⊗ – SEALED SAMPLES

⊙ – TESTED IMMEDIATELY

| SAMPLE | STRENGTH Kg/cm$^2$ | STRAIN | MOISTURE CONTENT TOP | MIDDLE | BOTTOM |
|---|---|---|---|---|---|
| 1 | 7.72 | 3.5% | 8.9% | 9.4% | 10.8% |
| 2 | 11.50 | 4.9 | 8.6 | 9.3 | 10.4 |
| 3 | 7.17 | 4.9 | 8.7 | 9.4 | 10.1 |
| 4 | 10.15 | 3.6 | 9.1 | 9.8 | 10.1 |
| 5 | 10.10 | 4.6 | 8.8 | 9.2 | 10.6 |
| x | | | 8.9 | 10.3 | 13.2 |

| TIME | WT. GMS. | DIAL RDG. | ΔH INS. | DENSITY | REMARKS |
|---|---|---|---|---|---|
| J3-3:35P | 11,096 | 0.0842 | 0.000 | 122.1 | Soak |
| 6-11:00A | 11,128 | .0923 | | | |
| 9-2:00P | 11,138 | .0960 | | | |
| 15-10:15A | 11,145 | .1005 | | | |
| 16-10:00A | 11,147 | .1008 | .017 | 121.9 | Freeze |
| 17-10:00A | | .0888 | .004 | 122.1 | Thaw |
| 18-3:30P | 11,146 | .1245 | .040 | 121.2 | Soak (1 CYCLE) |
| 20-11:30P | 11,151 | .1280 | | | |
| 21-1:00P | 11,153 | .1280 | | | |
| 24-12:00P | 11,155 | .1303 | .046 | 121.2 | Freeze |
| 25-12:00P | | .1148 | .031 | 121.6 | Thaw |
| 26-7:00P | 11,155 | .1420 | .058 | 121.0 | Soak (2 CYCLES) |
| 27-2:30P | 11,158 | .1490 | | | |
| 29-6:00P | 11,167 | .1492 | | | |
| A4-10:30A | 11,170 | .1505 | .066 | 121.0 | Freeze |
| 5-10:30A | | .1340 | .050 | 121.2 | Thaw |
| 6-11:00A | 11,172 | .1595 | .075 | 120.8 | Soak (3 CYCLES) |
| 7-2:30P | 11,176 | .1610 | | | |
| 11-9:00A | 11,186 | .1635 | .079 | 120.9 | Freeze |
| 12-9:00A | | .1465 | .062 | 121.0 | Thaw |
| 13-9:30A | 11,180 | .1718 | .088 | 120.2 | (4 CYCLES) |
| 13-12:30P | 11,229 | | | | Two samplers driven |
| 17-10:30A | 11,246 | | | | and sealed-Soak |
| 19-9:00A | 11,252 | | | | Tested |

CL MATERIAL TEST DATA - STANDARD PROCTOR DENSITY

| CYCLES | DENSITY | STRENGTH | STRAIN | W-TOP | W-MIDDLE | W-BOTTOM |
|---|---|---|---|---|---|---|
| 0 | *113.2#/cf | 1.20kg/cm$^2$ | 5.1% | 13.3% | 13.5% | 12.0% |
| 1/3 | 113.4 | 1.76 | 5.6 | 13.2 | 13.5 | 13.5 |
| | | 1.67 | 5.3 | | 13.4 | |
| | | *2.03 | 4.0 | 13.1 | 12.0 | 13.2 |
| | | *1.95 | 3.0 | 13.1 | 13.2 | 13.4 |
| Av. W.-14.0% | | At center of mold - | | 13.2 | 13.8 | 14.2 |
| 2/3 | 113.3 | 1.20 | 2.5 | 14.1 | 14.2 | 14.2 |
| 1 | *113.4 | 1.19 | 5.3 | 14.5 | 14.0 | 14.1 |
| 1 1/3 | 113.4 | 0.98 | 2.7 | | 14.3 | |
| | | *0.98 | 2.5 | 14.3 | 14.5 | 14.6 |
| | | *0.74 | 2.6 | 14.5 | 15.5 | 16.1 |
| Av. W.-15.3% | | At center of mold - | | 14.9 | 13.9 | 15.5 |
| 1 2/3 | 112.7 | 1.31 | 3.6 | 13.5 | 13.6 | 10.9 |
| 2 | *113.4 | 1.19 | 5.4 | 13.8 | 13.5 | 13.6 |
| 2 1/3 | 113.4 | 1.30 | 4.2 | | 14.0 | |
| | | *0.98 | 2.3 | 15.0 | 14.9 | 14.8 |
| | | *1.47 | 3.4 | 13.3 | 13.5 | 13.4 |
| Av.W. -16.2% | | At center of mold - | | 14.2 | 14.0 | 13.5 |
| 2 2/3 | 112.2 | 0.85 | 6.1 | 15.1 | 14.5 | 14.1 |
| 3 | *113.4 | 0.98 | 2.8 | 15.1 | 14.5 | 14.2 |
| 3 1/3 | 113.2 | 0.84 | 2.4 | | 14.9 | |
| | | *0.98 | 2.3 | 11.7 | 14.5 | 14.4 |
| | | *1.17 | 3.3 | 14.5 | 14.7 | 14.2 |
| Av. W.-15.9% | | At center of mold - | | 15.0 | 14.2 | 14.2 |
| 3 2/3 | 111.7 | 1.66 | 5.5 | 14.4 | 14.2 | 14.0 |
| 4 | *113.0 | 0.49 | 2.4 | | 15.0 | |
| 4 1/3 | 113.0 | 0.98 | 2.4 | 15.3 | 14.3 | 14.3 |
| | | *1.24 | 1.9 | 14.0 | 14.0 | 13.7 |
| | | *1.45 | 3.7 | 14.2 | 14.2 | 13.9 |
| Av. W.-15.9% | | At center of mold - | | 14.7 | 14.0 | 14.5 |
| 4 2/3 | 111.3 | | | | | |
| 5 | 113.0 | | | | | |
| 5 1/3 | 113.0 | | | | | |
| 8 2/3 | 109.9 | 1.05 | 4.6 | 15.3 | 14.9 | 15.3 |
| 9 | *111.8 | 1.06 | 4.5 | 15.7 | 15.2 | 11.9 |
| 9 1/3 | | 0.87 | 3.8 | 16.0 | 15.0 | 14.7 |
| | | *0.95 | 6.1 | 15.7 | 15.3 | 15.4 |
| | | *0.68 | 3.6 | 15.0 | 14.8 | 14.9 |
| Av. W--16.7% | | At center of mold - | | 15.0 | 14.2 | 15.1 |

CL MATERIAL TEST DATA - MODIFIED PROCTOR DENSITY

| CYCLES | DENSITY | STRENGTH | STRAIN | W-TOP | W-MIDDLE | W-BOTTOM |
|---|---|---|---|---|---|---|
| 0 | *122.4#/cf | 2.38kg/cm$^2$ | 4.0% | | 11.2% | |
| 1/3 | 122.4 | 5.50 | 4.7 | | 10.8 | |
| | | 5.06 | 4.3 | | 10.6 | |
| | | *5.90 | 2.5 | | 10.0 | |
| Av. W.-10.3% | | *5.37 | 3.2 | | 10.3 | |
| | | | | | | |
| 2/3 | 122.4 | 3.52 | 6.5 | | 10.7 | |
| 1 | *122.3 | 4.77 | 5.0 | | 11.0 | |
| 1 1/3 | 122.3 | 2.89 | 5.3 | | 10.9 | |
| | | *5.17 | 2.3 | | 10.7 | |
| Av. W.-11.3% | | *4.40 | 3.0 | | 10.5 | |
| | | | | | | |
| 1 2/3 | 122.2 | 2.40 | 4.7 | | 11.9 | |
| 2 | *122.2 | 2.56 | 2.2 | | 11.6 | |
| 2 1/3 | 122.2 | 1.21 | 3.8 | | 11.9 | |
| | | *4.26 | 3.2 | | 11.0 | |
| | | *3.67 | 2.8 | | 11.1 | |
| Av. W.-12.0% | | At center of mold - | | 11.3% | 11.8 | 12.1% |
| | | | | | | |
| 2 2/3 | 122.0 | 2.29 | 3.0 | | 12.3 | |
| 3 | *122.0 | 1.93 | 3.9 | | 12.0 | |
| 3 1/3 | 121.9 | 1.90 | 5.4 | | 12.2 | |
| | | *2.94 | 2.6 | | 11.3 | |
| | | *3.15 | 3.5 | | 11.5 | |
| Av. W.-12.6% | | At center of mold - | | 11.2 | 11.7 | 12.8 |
| | | | | | | |
| 3 2/3 | 121.3 | 2.62 | 5.2 | 11.4 | 11.9 | 12.4 |
| 4 | *121.9 | 1.67 | 5.1 | | 12.0 | |
| 4 1/3 | 121.9 | 2.12 | 6.2 | 11.1 | 11.7 | 13.0 |
| | | *3.42 | 2.8 | 10.9 | 11.1 | 11.6 |
| | | *2.90 | 3.9 | 11.4 | 11.5 | 11.8 |
| Av. W.-13.0% | | At center of mold - | | 13.5 | 12.0 | 11.4 |
| | | | | | | |
| 4 2/3 | 120.8 | | | | | |
| 5 | 121.6 | | | | | |
| 5 1/3 | 121.4 | | | | | |
| | | | | | | |
| 8 2/3 | 119.0 | 2.62 | 5.0 | 13.3 | 12.2 | 12.0 |
| 9 | *120.4 | 1.92 | 4.8 | 12.8 | 12.4 | 11.9 |
| 9 1/3 | | 2.41 | 4.4 | | 12.7 | |
| | | *2.93 | 2.9 | 13.0 | 12.2 | 11.6 |
| | | *2.96 | 1.9 | 12.8 | 12.4 | 11.6 |
| Av. W.-13.9% | | At center of mold - | | 14.1 | 12.5 | 11.9 |

CI (1) MATERIAL TEST DATA - STANDARD PROCTOR DENSITY

| CYCLES | DENSITY | STRENGTH | STRAIN | W-TOP | W-MIDDLE | W-BOTTOM |
|---|---|---|---|---|---|---|
| 0 | *108.4#/cf | 1.43kg/cm$^2$ | 5.1% | 15.7% | 14.6% | 14.3% |
| 1/3 | 108.2 | 1.38 | 8.9 | 14.5 | 14.3 | 16.1 |
| | | 2.20 | 4.8 | | 14.4 | |
| | | *2.17 | 3.9 | 14.2 | 14.8 | 14.3 |
| | | | | 13.2 | 13.8 | 13.5 |
| Av. W. -15.4% | | At center of mold - | | 13.9 | 14.5 | 15.3 |
| 2/3 | 108.1 | 1.51 | 6.0 | 16.3 | 15.6 | 15.3 |
| 1 | *108.1 | 1.17 | 6.7 | 16.7 | 15.4 | 15.5 |
| 1 1/3 | 107.9 | 1.22 | 3.9 | 16.0 | 15.2 | 15.2 |
| | | *1.69 | 4.3 | 15.6 | 15.1 | 15.5 |
| | | *1.64 | 6.6 | 15.8 | 15.3 | 15.8 |
| Av. W. -15.9% | | At center of mold - | | 15.3 | 15.5 | 16.0 |
| 1 2/3 | 107.7 | 1.27 | 6.5 | 15.0 | 15.6 | 16.9 |
| 2 | *107.7 | 1.04 | 6.4 | 14.8 | 15.8 | 16.8 |
| 2 1/3 | 107.8 | 1.45 | 4.2 | | 15.2 | |
| | | *1.29 | 5.0 | 15.3 | 15.4 | 16.1 |
| | | *1.31 | 4.0 | 15.6 | 15.5 | 16.1 |
| Av. W. -16.2% | | At center of mold - | | 14.7 | 15.9 | 17.7 |
| 2 2/3 | 107.3 | 0.96 | 4.5 | 15.7 | 16.5 | 17.6 |
| 3 | *107.6 | 1.02 | 8.1 | 15.6 | 16.4 | 17.7 |
| 3 1/3 | 107.5 | 1.15 | 8.4 | | 16.5 | |
| | | *1.18 | 6.4 | 15.9 | 16.4 | 17.1 |
| | | *1.49 | 7.5 | 15.8 | 16.3 | 16.8 |
| Av. W. -17.0% | | At center of mold - | | 15.8 | 16.8 | 17.7 |
| 3 2/3 | 107.0 | 0.94 | 6.4 | 15.5 | 16.9 | 18.1 |
| 4 | *107.2 | 1.03 | 7.2 | 15.0 | 16.5 | 17.4 |
| 4 1/3 | 107.2 | 1.07 | 7.9 | | 16.6 | |
| | | *1.32 | 6.4 | 15.7 | 16.5 | 17.2 |
| | | *1.29 | 8.3 | 15.8 | 16.7 | 17.6 |
| Av. W. -16.9% | | At center of mold - | | 14.9 | 16.7 | 17.9 |
| 4 2/3 | 106.7 | | | | | |
| 5 | 107.0 | | | | | |
| 5 1/3 | 106.9 | | | | | |
| 8 2/3 | 104.0 | 0.41 | 8.7 | 19.5 | 19.9 | 19.0 |
| 9 | *105.1 | 0.54 | 10.0 | 19.1 | 19.5 | 18.9 |
| 9 1/3 | | 0.63 | 12.2 | 18.9 | 19.6 | 19.2 |
| | | *0.56 | 8.0 | 18.8 | 18.8 | 19.4 |
| | | *0.56 | 7.0 | 19.2 | 19.0 | 19.2 |
| Av. W. -19.6% | | At center of mold - | | 19.2 | 19.3 | 19.1 |

## CI (1) MATERIAL TEST DATA - MODIFIED PROCTOR DENSITY

| CYCLES | DENSITY | STRENGTH | STRAIN | W-TOP | W-MIDDLE | W-BOTTOM |
|---|---|---|---|---|---|---|
| 0 | *122.1#/cf | 12.07kg/cm$^2$ | 3.9% | | 9.0% | |
| 1/3 | 121.8 | 8.75 | 5.3 | | 9.5 | |
| | | 8.28 | 3.1 | | 9.1 | |
| | | *11.20 | 3.3 | | 8.7 | |
| Av. W. -9.9% | | *17.40 | 3.3 | | 8.4 | |
| 2/3 | 122.0 | 7.63 | 5.2 | | 9.2 | |
| 1 | *121.1 | 7.85 | 5.2 | | 9.5 | |
| 1 1/3 | 121.0 | 8.34 | 4.7 | | 9.2 | |
| | | *10.70 | 3.0 | | 9.4 | |
| | | *11.41 | 3.6 | | 9.6 | |
| Av. W.-10.2% | | At center of mold - 9.0% | | | 9.2 | 11.7% |
| 1 2/3 | 121.2 | 7.08 | 6.3 | | 9.9 | |
| 2 | *120.8 | 5.68 | 5.9 | | 10.4 | |
| 2 1/3 | 120.6 | 9.15 | 4.3 | | 9.6 | |
| | | *11.10 | 4.1 | | 9.9 | |
| | | *10.68 | 3.6 | | 9.8 | |
| Av. W.-10.9% | | At center of mold - 9.1 | | | 10.6 | 13.0 |
| 3 2/3 | 121.0 | 7.72 | 3.5 | 8.9 | 9.4 | 10.8 |
| 4 | *120.2 | 11.50 | 4.9 | 8.6 | 9.3 | 10.4 |
| 4 1/3 | 120.1 | 7.17 | 4.9 | 8.7 | 9.4 | 10.1 |
| | | *10.15 | 3.6 | 9.1 | 9.8 | 10.1 |
| | | *10.10 | 4.6 | 8.8 | 9.2 | 10.8 |
| Av. W.-10.3% | | At center of mold - 8.9 | | | 10.3 | 13.2 |
| 5 2/3 | 120.0 | 5.70 | 4.7 | 8.4 | 8.8 | 10.9 |
| 6 | *119.7 | 6.32 | 4.8 | 8.5 | 9.9 | 12.2 |
| 6 1/3 | 119.7 | 7.32 | 3.3 | | 8.7 | |
| | | *6.78 | 3.7 | 8.4 | 9.7 | 11.3 |
| | | *11.46 | 3.1 | 8.6 | 9.3 | 10.7 |
| Av. W.-11.0% | | At center of mold - 8.3 | | | 10.0 | 13.8 |
| 8 2/3 | 118.3 | 4.42 | 7.5 | 10.2 | 12.6 | 14.0 |
| 9 | *118.1 | 3.29 | 6.3 | 10.4 | 13.3 | 14.0 |
| 9 1/3 | | 4.07 | 10.0 | 10.2 | 12.8 | 14.3 |
| | | *7.52 | 6.4 | 10.8 | 11.2 | 10.1 |
| | | *5.96 | 8.9 | 10.5 | 11.9 | 13.2 |
| Av. W.-12.5% | | At center of mold - 9.3 | | | 12.2 | 13.7 |

## CI (2) MATERIAL TEST DATA - STANDARD PROCTOR DENSITY

| CYCLES | DENSITY | STRENGTH | STRAIN | W-TOP | W-MIDDLE | W-BOTTOM |
|---|---|---|---|---|---|---|
| 0 | 103.3#/cf | 1.87kg/cm$^2$ | 11.5% | | 19.9% | |
| 1/3 | *103.0 | 1.85 | 12.1 | | 19.9 | |
| | | 1.91 | 9.4 | | 20.0 | |
| | | *2.29 | 12.3 | | 19.2 | |
| | | *2.40 | 8.1 | | 19.9 | |
| Av. W.-20.2% | | At center of mold - | | 19.8% | 19.6 | 20.1% |
| 2/3 | 102.3 | 1.10 | 13.0 | | 20.6 | |
| 1 | *102.5 | 0.99 | 10.1 | | 20.6 | |
| 1 1/3 | 102.3 | 0.78 | 9.0 | | 20.6 | |
| | | *1.24 | 12.2 | | 20.1 | |
| | | *1.09 | 9.6 | | 20.1 | |
| Av. W.-21.2% | | At center of mold - | | 20.0 | 20.9 | 21.4 |
| 1 2/3 | 100.0 | 0.88 | 12.0 | | 21.3 | |
| 2 | *101.4 | 0.78 | 8.5 | 21.6 | 21.4 | 21.3 |
| 2 1/3 | 101.2 | 0.82 | 8.6 | 21.2 | 21.1 | 21.4 |
| | | *0.89 | 7.1 | 20.6 | 21.0 | 20.4 |
| Av. W.-21.6% | | *1.28 | 14.2 | 20.7 | 19.9 | 20.6 |
| | | | | 20.2 | 21.1 | 21.4 |
| 2 2/3 | 99.3 | | | | 23.4 | |
| 3 | *100.8 | 0.25 | | 23.1 | 23.1 | 23.7 |
| 3 1/3 | 100.7 | 0.46 | 8.6 | | 23.2 | |
| | | *0.54 | 10.4 | 22.6 | 22.7 | 22.8 |
| | | *0.47 | 6.5 | | 23.1 | |
| Av. W.-23.4% | | At center of mold - | | 23.9 | 22.5 | 22.5 |
| 3 2/3 | 98.5 | 0.24 | 2.6 | 24.0 | 23.0 | 19.4 |
| 4 | *100.3 | 0.45 | 9.9 | 23.5 | 23.1 | 22.0 |
| 4 1/3 | 100.2 | 0.34 | 4.4 | 22.8 | 22.8 | 22.8 |
| | | *0.72 | 13.2 | 22.5 | 22.5 | 22.0 |
| | | *0.80 | 15.7 | 22.7 | 22.5 | 21.9 |
| Av. W.-23.9% | | At center of mold - | | 23.5 | 22.4 | 22.0 |
| 4 2/3 | 98.2 | | | | | |
| 5 | 100.1 | | | | | |
| 5 1/3 | 99.8 | | | | | |
| 8 1/3 | 95.1 | 0.51 | 14.5 | 24.5 | 23.9 | 23.0 |
| 9 | *97.6 | 0.30 | 14.1 | 24.7 | 24.9 | 23.2 |
| 9 1/3 | | 0.46 | 21.4 | 24.8 | 21.3 | 23.9 |
| | | *0.53 | 12.2 | 23.9 | 23.6 | 23.4 |
| | | *0.63 | 23.7 | 22.9 | 23.4 | 23.5 |
| Av. W.-25.1% | | At center of mold - | | 24.7 | 23.1 | 23.3 |

## CI (2) MATERIAL TEST DATA - MODIFIED PROCTOR DENSITY

| CYCLES | DENSITY | STRENGTH | STRAIN | W-TOP | W-MIDDLE | W-BOTTOM |
|---|---|---|---|---|---|---|
| 0 | *119.4#/cf | 10.58kg/cm$^2$ | 6.1% | | 13.0% | |
| 1/3 | 117.7 | 10.67 | 7.8 | | 12.9 | |
| | | 8.10 | 5.4 | | 13.2 | |
| | | *10.15 | 3.5 | | 12.8 | |
| Av. W.-13.3% | | *11.20 | 8.9 | | 12.4 | |
| 2/3 | 117.7 | 7.70 | 7.0 | | 13.8 | |
| 1 | *117.6 | 7.75 | 8.1 | | 14.1 | |
| 1 1/3 | 117.4 | 7.40 | 8.7 | | 14.1 | |
| | | * 10.17 | 8.9 | | 13.7 | |
| | | *10.18 | 10.0 | | 13.1 | |
| Av. W.-14.3% | | At center of mold - 12.8% | | | 14.0 | 16.3% |
| 1 2/3 | 117.1 | 7.14 | 8.5 | | 14.1 | |
| 2 | *117.0 | 6.80 | 6.4 | | 14.2 | |
| 2 1/3 | 116.9 | 6.83 | 9.4 | | 14.2 | |
| | | *9.90 | 8.5 | | 13.5 | |
| | | *9.60 | 7.1 | | 13.5 | |
| Av. W.-14.6% | | At center of mold - 13.0 | | | 14.0 | 15.8 |
| 2 2/3 | 116.5 | 6.78 | 9.8 | 13.6 | 14.3 | 15.4 |
| 3 | *116.7 | 6.00 | 7.7 | 13.4 | 14.3 | 15.5 |
| 3 1/3 | 116.6 | 6.85 | 9.5 | | 14.7 | |
| | | *9.18 | 8.6 | 13.7 | 14.0 | 14.8 |
| | | *9.17 | 8.9 | 13.5 | 14.1 | 15.0 |
| Av. W.-14.8% | | At center of mold - 13.2 | | | 14.6 | 16.8 |
| 3 2/3 | 116.0 | 5.87 | 10.3 | 13.7 | 14.9 | 15.9 |
| 4 | *116.6 | 6.64 | 5.8 | | 15.0 | |
| 4 1/3 | 116.5 | 5.45 | 9.8 | | 15.1 | |
| | | *6.62 | 12.2 | 14.1 | 15.0 | 15.8 |
| | | *7.56 | 11.6 | | 14.8 | |
| Av. W.-14.5% | | At center of mold - 13.3 | | | 15.2 | 16.7 |
| 4 2/3 | 115.9 | | | | | |
| 5 | 116.3 | | | | | |
| 5 1/3 | 116.2 | | | | | |
| 8 2/3 | 114.7 | 5.16 | 13.2 | 14.8 | 16.7 | 17.4 |
| 9 | *115.1 | 4.58 | 13.6 | 14.2 | 15.1 | 17.4 |
| 9 1/3 | | 4.72 | 10.5 | 14.2 | 15.7 | 17.3 |
| | | *6.20 | 12.0 | 14.1 | 15.2 | 16.9 |
| | | *7.62 | 10.7 | 14.5 | 15.3 | 16.9 |
| Av. W.-15.8% | | At center of mold - 13.8 | | | 15.5 | 17.7 |

## CH MATERIAL TEST DATA - STANDARD PROCTOR DENSITY

| CYCLES | DENSITY | STRENGTH | STRAIN | W-TOP | W-MIDDLE | W-BOTTOM |
|---|---|---|---|---|---|---|
| 0 | *88.2#/cf | 1.43kg/cm | 5.3% | | 26.8% | |
| 1/3 | 86.3 | 1.45 | 4.1 | 25.7% | | 29.0% |
| | | 2.16 | 4.6 | 23.2 | 26.7 | 29.2 |
| | | *2.25 | 2.4 | 26.0 | 24.4 | 20.8 |
| | | *2.95 | 1.7 | 19.4 | 19.2 | 19.0 |
| Av. W.-26.1% | | At center of mold - | | 21.7 | 27.6 | 26.8 |
| 2/3 | 86.7 | 1.17 | 6.9 | 27.6 | 29.4 | 31.4 |
| 1 | *86.1 | 1.17 | 7.2 | 27.0 | 29.5 | 31.2 |
| 1 1/3 | 85.4 | 1.16 | 8.1 | | 30.0 | |
| | | *1.92 | 3.8 | 26.7 | 24.2 | 23.5 |
| | | *3.25 | 6.9 | 23.5 | 25.8 | 27.8 |
| Av. W.-29.1% | | At center of mold - | | 25.7 | 29.4 | 36.8 |
| 1 2/3 | 86.1 | 0.84 | 12.0 | 31.2 | 31.4 | 33.2 |
| 2 | *85.5 | 0.71 | 6.0 | 31.5 | 30.7 | 32.9 |
| 2 1/3 | 85.2 | 0.85 | 10.2 | | 32.1 | |
| | | *1.22 | 2.3 | 25.1 | 25.1 | 26.6 |
| | | *0.98 | 2.9 | 23.5 | 25.5 | 26.1 |
| Av. W.-31.9% | | At center of mold - | | 30.7 | 31.4 | 33.6 |
| 2 2/3 | 85.5 | 0.46 | 8.9 | 36.0 | 35.1 | 35.0 |
| 3 | *85.2 | 0.38 | 8.5 | 36.8 | 34.5 | 35.3 |
| 3 1/3 | 84.4 | 0.36 | 9.8 | | 35.7 | |
| | | *1.13 | 2.5 | 35.9 | 28.7 | 28.5 |
| | | *0.93 | 7.1 | 36.2 | 27.7 | 28.9 |
| Av. W.-35.0% | | At center of mold - | | 35.0 | 33.3 | 35.0 |
| 3 2/2 | 84.6 | 0.33 | 6.2 | 36.4 | 35.2 | 37.9 |
| 4 | *84.3 | 0.37 | 8.9 | 37.0 | 35.4 | 37.3 |
| 4 1/3 | 83.6 | 0.35 | 13.3 | 36.4 | 35.6 | 38.0 |
| | | *0.94 | 7.1 | 31.3 | 31.5 | 31.4 |
| | | *0.57 | 4.2 | 32.2 | 31.9 | 32.7 |
| Av. W.-36.8% | | At center of mold - | | 37.5 | 35.1 | 35.4 |
| 4 2/3 | 83.2 | | | | | |
| 5 | 82.8 | | | | | |
| 5 1/3 | 82.3 | | | | | |
| 8 2/3 | 76.7 | 0.27 | 10.7 | 40.0 | 38.5 | 38.2 |
| 9 | *77.7 | 0.26 | 13.0 | 41.2 | 39.3 | 37.2 |
| 9 1/3 | | 0.26 | 14.3 | 36.2 | 39.2 | 38.5 |
| | | *0.29 | 4.5 | 37.7 | 36.8 | 36.7 |
| | | *0.29 | 3.9 | 38.3 | 38.1 | 37.8 |
| Av. W.-40.4% | | At center of mold - | | 40.2 | 37.3 | 38.6 |

## CH MATERIAL TEST DATA - MODIFIED PROCTOR DENSITY

| CYCLES | DENSITY | STRENGTH | STRAIN | W-TOP | W-MIDDLE | W-BOTTOM |
|---|---|---|---|---|---|---|
| 0 | *101.0#/cf | 6.46kg/cm$^2$ | 4.8% | | 19.6% | |
| 1/3 | 99.5 | 6.70 | 4.2 | | 19.6 | |
| | | 6.48 | 4.2 | | 20.0 | |
| | | *10.13 | 3.3 | | 17.3 | |
| Av. W.-20.7% | | *11.20 | 3.3 | | 17.4 | |
| 2/3 | 99.7 | 5.50 | 7.8 | | 22.0 | |
| 1 | *99.2 | 5.04 | 6.3 | | 21.5 | |
| 1 1/3 | 98.8 | 5.03 | 6.8 | | 21.7 | |
| | | *8.87 | 4.6 | | 20.1 | |
| | | *6.66 | 5.3 | | 20.7 | |
| Av. W.-22.5% | | At center of mold - | | 18.7% | 21.6 | 25.2% |
| 1 2/3 | 98.8 | 5.25 | 9.6 | | 23.5 | |
| 2 | *98.7 | 5.20 | 10.0 | | 23.3 | |
| 2 1/3 | 98.2 | 5.22 | 9.3 | | 23.0 | |
| | | *7.05 | 6.6 | | 21.8 | |
| | | *6.48 | 7.0 | | 21.9 | |
| Av. W.-23.5% | | At center of mold - | | 20.5 | 23.5 | 26.2 |
| 2 2/3 | 98.2 | 4.44 | 6.8 | 20.4 | 23.2 | 25.1 |
| 3 | *98.2 | 5.13 | 7.1 | 19.9 | 22.1 | 24.4 |
| 3 1/3 | 97.9 | 4.67 | 7.1 | 19.9 | 22.2 | 24.8 |
| | | *5.72 | 5.6 | 20.0 | 23.4 | 22.0 |
| | | *6.62 | 5.9 | 19.6 | 21.0 | 22.7 |
| Av. W.-22.8% | | At center of mold - | | 20.0 | 23.8 | 26.6 |
| 3 2/3 | 97.7 | | | | | |
| 4 | 97.7 | | | | | |
| 4 1/3 | 97.6 | | | | | |
| 4 2/3 | 97.2 | | | | | |
| 5 | 97.4 | | | | | |
| 5 1/3 | 97.3 | | | | | |
| 5 2/3 | 96.9 | 3.70 | 7.9 | 22.5 | 24.5 | 25.6 |
| 6 | *97.0 | 3.27 | 6.9 | 23.1 | 25.0 | 25.9 |
| 6 1/3 | 97.0 | 3.89 | 8.8 | | 24.6 | |
| | | *5.47 | 9.2 | 22.0 | 23.5 | 23.8 |
| | | *5.58 | 7.3 | 22.1 | 23.4 | 24.2 |
| Av. W.-24.5% | | At center of mold - | | 21.6 | 24.7 | 26.6 |
| 8 2/3 | 94.8 | 3.20 | 9.2 | 25.9 | 26.2 | 27.0 |
| 9 | *95.4 | 3.55 | 11.5 | 25.6 | 26.4 | 26.7 |
| 9 1/3 | | 3.62 | 9.5 | 25.1 | 26.2 | 25.9 |
| | | *4.28 | 5.4 | 23.2 | 23.8 | 24.7 |
| | | *4.27 | 6.0 | 23.5 | 24.5 | 25.0 |
| Av. W.-26.5% | | At center of mold - | | 25.2 | 25.9 | 26.3 |

# APPENDIX B

## Summary of Granular Soil Test Data

The test data for the granular soil has been summarized from the test data sheets of which a typical data sheet is shown on the following page. The results of each specimen has been summarized under the following headings:

Cycles - Number of cycles to which the specimen has been subjected.

Density - Average density existing for a given number of cycles.

Strength - The strength values have been recorded under the major and minor principal stress for each sample which was tested.

Strain - Corresponding to the strength values.

W-top and bottom - Water contents found in the top and bottom half of the samples.

The discrepencies between the results as summarized and those which should be recorded according to the test procedure which was outlined are due to the variations in testing procedure until it was standardized.

| UNIVERSITY OF ALBERTA | |
|---|---|
| MOLD NO. 7 WT., GMS. | 7,342 |
| WT. SAMPLE WET + MOLD, GMS. | 14,880 |
| WT. SAMPLE WET GMS. | 7,538 |
| VOLUME MOLD, CF. | 0.1143 |
| AVE. MOISTURE CONTENT | 7.1 |
| WET UNIT WT., LB/FT$^3$ | 145.0 |
| DRY UNIT WT. LB/FT$^3$ | 135.3 |

UNIT WEIGHT DETERMINATION

MOISTURE CONTENT ON COMPACTION

| LAYER | MOISTURE CONTENT |
|---|---|
| | |
| | |
| AVERAGE | 7.1 |
| AVERAGE MOISTURE CONT. AT END = | 8.4 |
| % SATURATION AT START = | |
| % SATURATION AT END = | |

## MOISTURE DENSITY AND STRENGTH DATA

DATE August 18, 1959

MATERIAL Granular Soil

COMPACTION Modified Proctor

UNCONFINED COMPRESSION TEST RESULTS

30° Kg/cm$^2$

| SAMPLE | STRENGTH | STRAIN | MOISTURE CONTENT TOP | MIDDLE | BOTTOM |
|---|---|---|---|---|---|
| 1 | $\sigma_1$ = 2.32 | 3.8% | $\sigma_3$ | = 0.00 | Kg/cm$^2$ |
| 2 | = 4.27 | 3.2 | | = 0.42 | |
| 3 | = 4.82 | 2.4 | | = 0.62 | |
| 4 | = 5.75 | 2.5 | | = 0.93 | |
| 5 | = 2.60 | 1.0 | | = 0.00 | |
| x | | | 7.9 | | 8.8 |

| TIME | WT. GMS. | DIAL RDG. | $\Delta$H INS. | DENSITY | REMARKS |
|---|---|---|---|---|---|
| A18-2:30P | 15,041 | 0.0150 | 0.000 | 134.2 | Soak |
| 19-9:40A | 15,049 | .0150 | | | |
| 25-2:20P | 15,054 | .0158 | | | |
| 27-11:00A | 15,051 | .0158 | .000 | 134.2 | Freeze |
| 28-10:00A | | .0172 | .002 | 134.1 | Thaw |
| 29-10:00A | 15,051 | .0200 | .005 | 134.1 | Soak (1 CYCLE) |
| S1-1:30P | 15,061 | .0200 | .005 | 134.1 | |
| 2-3:30P | 15,066 | .0190 | .004 | 134.1 | Freeze |
| 3-4:00P | | .0248 | .010 | 134.0 | Thaw |
| 4-4:00P | 15,057 | .0278 | .013 | 134.0 | Soak (2 CYCLES) |
| 8-11:30A | 15,060 | .0277 | .013 | 134.0 | Freeze |
| 9-1:00P | | .0370 | .022 | 133.8 | Thaw |
| 10-2:00P | 15,065 | .0372 | .022 | 133.8 | Soak (3 CYCLES) |
| 14-10:00A | 15,070 | .0376 | .023 | 133.8 | Freeze |
| 15-12:00P | | .0468 | .032 | 133.6 | Thaw |
| 16-1:30P | 15,070 | .0468 | .032 | 133.6 | Soak (4 CYCLES) |
| 20-9:30P | 15,084 | .0475 | | | |
| 22-5:00P | 15,090 | .0478 | .033 | 133.6 | Freeze |
| 23-10:00P | | .0639 | .049 | 133.1 | Thaw |
| 24-11:00P | 15,086 | .0630 | .048 | 133.1 | Soak (5 CYCLES) |
| 27-2:00P | 15,105 | .0640 | .049 | 133.1 | |
| 30-7:30P | 15,100 | .0650 | .050 | 133.1 | |
| O4-5:00P | 15,110 | .0655 | .051 | 133.1 | Freeze |
| 5-7:00P | | .0865 | .072 | 132.6 | Thaw |
| 6-7:00P | 15,109 | .0825 | .068 | 132.6 | Tested (6 CYCLES) |

GRAVEL TEST DATA - STANDARD PROCTOR DENSITY

| CYCLES | DENSITY | $^*\sigma_1$ kg/cm2 | $^*\sigma_3$ kg/cm2 | STRAIN | W-TOP | W-BOTTOM |
|---|---|---|---|---|---|---|
| 0 | 121.0#/cf | 3.31 | 0.21 | 2.3% | 7.8 | 7.8 |
| | | 4.00 | 0.41 | 2.3 | | |
| | | 4.31 | 0.62 | 2.5 | | |
| | | 4.89 | 0.83 | 3.1 | | |
| | | 3.24 | 0.21 | 0.5 | | |
| 1/3 | 119.9 | 1.95 | 0.21 | 2.9 | 9.2 | 11.0 |
| | | 2.53 | 0.41 | 2.1 | | |
| | | 3.22 | 0.62 | 3.1 | | |
| | | 3.85 | 0.83 | 3.6 | | |
| | | 2.31 | 0.21 | 1.6 | | |
| 2/3 | 119.0 | | | | | |
| 1 | 119.1 | | | | | |
| 1 1/3 | 119.2 | | | | | |
| 1 2/3 | 117.8 | 1.10 | 0.21 | 3.5 | 11.2 | 12.2 |
| 2 | 118.2 | 1.59 | 0.42 | 4.0 | | |
| | | 1.98 | 0.63 | 4.0 | | |
| | | 2.21 | 0.83 | 3.6 | | |
| | | 1.36 | 0.21 | 7.0 | | |
| 2 1/3 | 117.3 | 1.15 | 0.21 | 6.3 | 12.1 | 12.2 |
| | | 1.63 | 0.41 | 4.1 | | |
| | | 1.98 | 0.62 | 3.1 | | |
| | | 2.21 | 0.83 | 3.4 | | |
| | | 1.33 | 0.21 | 1.3 | | |
| 2 2/3 | 115.5 | 0.62 | 0.21 | 3.6 | 13.1 | 13.5 |
| 3 | 116.9 | 0.88 | 0.31 | 1.2 | | |
| 3 1/3 | 117.0 | 1.46 | 0.83 | 3.3 | | |
| 3 2/3 | 115.6 | | | | | |
| 4 | 116.5 | | | | | |
| 4 1/3 | 116.5 | | | | | |
| 4 2/3 | 114.2 | | | | | |
| 5 | 116.1 | | | | | |
| 5 1/3 | 116.1 | | | | | |
| 5 2/3 | 113.5 | 0.49 | 0.20 | 4.3 | 14.3 | 14.6 |
| 6 | 115.5 | 0.91 | 0.52 | 6.9 | | |
| 6 1/3 | 115.5 | 1.45 | 0.94 | 6.9 | | |
| | | 0.37 | 0.00 | 2.0 | | |

$^*\sigma_1$ Major principal stress
$\sigma_3$ Minor principal stress

## GRAVEL TEST DATA - MODIFIED PROCTOR DENSITY

| CYCLES | DENSITY | * $\sigma_1$ kg/cm | * $\sigma_3$ kg/cm | STRAIN | W-TOP | W-BOTTOM |
|---|---|---|---|---|---|---|
| 0 | 134.2#/cf | 7.40 | 0.00 | 2.8% | 7.2 | 7.2 |
| | | 6.93 | 0.21 | 0.6 | | |
| | | 7.75 | 0.90 | 0.7 | | |
| 1/3 | 134.2 | 4.77 | 0.21 | 1.4 | 6.9 | 7.0 |
| | | 6.11 | 0.41 | 0.3 | | |
| | | 7.39 | 0.62 | 1.0 | | |
| | | 7.94 | 0.83 | 1.4 | | |
| | | 7.19 | 0.52 | 0.6 | | |
| 2/3 | 134.1 | 4.52 | 0.21 | 4.3 | 7.8 | 7.7 |
| 1 | 134.1 | 6.03 | 0.62 | 1.7 | | |
| | | 6.62 | 0.91 | 4.5 | | |
| | | 3.69 | 0.21 | 1.1 | | |
| | | 2.34 | 0.00 | 2.1 | | |
| 1 | 134.1 | 3.30 | 0.00 | 2.8 | 7.5 | 8.2 |
| | | 5.02 | 0.40 | 2.9 | | |
| | | 6.58 | 0.91 | 2.7 | | |
| | | 5.74 | 0.62 | 1.7 | | |
| | | 3.57 | 0.21 | 0.5 | | |
| 1 1/3 | 134.1 | 5.10 | 0.21 | 5.0 | 7.7 | 8.3 |
| | | 5.41 | 0.41 | 0.6 | | |
| | | 5.78 | 0.62 | 1.5 | | |
| | | 2.70 | 0.00 | - | | |
| 1 2/3 | 134.0 | 2.75 | 0.00 | 3.9 | 7.9 | 8.3 |
| 2 | 134.0 | 4.70 | 0.40 | 3.7 | | |
| | | 5.50 | 0.66 | 2.1 | | |
| | | 6.34 | 0.94 | 1.9 | | |
| | | 5.58 | 0.66 | 2.0 | | |
| 2 1/3 | 134.0 | 3.34 | 0.00 | 3.4 | 7.8 | 8.2 |
| | | 3.92 | 0.21 | 2.0 | | |
| | | 4.48 | 0.41 | 2.4 | | |
| | | 4.96 | 0.64 | 1.5 | | |
| | | 5.38 | 0.93 | 1.8 | | |
| 2 2/3 | 133.7 | 4.03 | 0.00 | 3.3 | 7.2 | 8.6 |
| 3 | 133.8 | 4.64 | 0.21 | 1.5 | | |
| | | 5.10 | 0.43 | 1.8 | | |
| | | 5.56 | 0.62 | 2.1 | | |
| | | 6.37 | 0.94 | 2.9 | | |

Continued

GRAVEL TEST DATA - MODIFIED PROCTOR DENSITY

| CYCLES | DENSITY | $*\sigma_1$ kg/cm$^2$ | $*\sigma_3$ kg/cm$^2$ | STRAIN | W-TOP | W-BOTTOM |
|---|---|---|---|---|---|---|
| 3 1/3 | 133.7#/cf | 2.23 | 0.00 | 3.6% | 7.9 | 8.4 |
| | | 2.98 | 0.20 | 2.4 | | |
| | | 3.79 | 0.41 | 2.3 | | |
| | | 4.49 | 0.62 | 2.3 | | |
| | | 5.32 | 0.92 | 2.8 | | |
| 3 2/3 | 133.5 | 2.32 | 0.00 | 2.6 | 6.9 | 8.1 |
| 4 | 133.6 | 3.57 | 0.22 | 2.9 | | |
| | | 4.24 | 0.41 | 2.2 | | |
| | | 4.88 | 0.62 | 2.3 | | |
| | | 5.81 | 0.95 | 3.1 | | |
| 4 1/3 | 133.5 | 2.88 | 0.00 | 3.7 | 7.9 | 8.8 |
| 4 2/3 | 133.1 | 3.67 | 0.21 | 2.1 | | |
| 5 | 133.1 | 4.58 | 0.40 | 2.1 | | |
| | | 5.26 | 0.63 | 3.1 | | |
| | | 6.03 | 0.91 | 2.5 | | |
| 5 1/3 | 133.1 | 2.32 | 0.00 | 3.8 | 7.9 | 8.8 |
| 5 2/3 | 132.6 | 4.27 | 0.42 | 3.2 | | |
| 6 | 132.6 | 4.82 | 0.62 | 2.4 | | |
| | | 5.75 | 0.93 | 2.5 | | |
| | | 2.60 | 0.00 | 1.0 | | |

$*\sigma_1$ - Major principal stress
$*\sigma_3$ - Minor principal stress

# A P P E N D I X   C

Summary of Calculations

The average water content, average void ratio, average strength, and the average degree of saturation as recorded in Appendix C were calculated as follows:

Average Water Content - The average water content was calculated from the increase in the weight of water in the sample. For each series of tests, the average value of the average water content for the respective soil samples at the same cycle was recorded.

Average Void Ratio - The average void ratio (e) was calculated for each of the samples from the following:

$$e = \frac{G \gamma_w V}{W_s} - 1$$

in which $G$ = Specific gravity of soil

$\gamma_w$ = Unit weight of water

$V$ = Volume of soil sample

$W_s$ = Weight of soil

For each series of tests, the average value of the average void ratio for the respective soil samples at the same cycle was recorded.

Average Strength - For each of the subgrade soils, the average of the unconfined compression strengths (Appendix A) at each of the saturation by capillarity cycles of testing were recorded.

Average Degree of Saturation - The average degree of saturation (S) was calculated for each of the samples from the following:

$$S = \frac{WG}{e}$$

in which W = water content.

For each series of tests, the average value of the average degree of saturation for the respective soil samples at the same cycle was recorded.

CL SOIL - STANDARD PROCTOR DENSITY SERIES

| CYCLES | *AV. w | *AV. e | *AV. STRENGTH | *AV. S |
|---|---|---|---|---|
| 0 | 13.5% | 0.502 | 1.99 kg/cm$^2$ | 73.5% |
| 1/3 | 13.8 | 0.500 | 1.58 | 75.4 |
| 1 1/3 | 14.2 | 0.496 | 1.12 | 78.2 |
| 2 1/3 | 14.6 | 0.498 | 1.27 | 80.0 |
| 3 1/3 | 14.9 | 0.507 | 0.89 | 80.2 |
| 4 1/3 | 15.3 | 0.503 | 1.04 | 83.0 |
| 9 | 15.8 | 0.522 | | 82.7 |
| 9 1/3 | | | 0.99 | |

CL SOIL - MODIFIED PROCTOR DENSITY SERIES

| | | | | |
|---|---|---|---|---|
| 0 | 10.5 | 0.405 | 5.65 | 70.8 |
| 1/3 | 11.9 | 0.405 | 4.31 | 80.2 |
| 1 1/3 | 12.5 | 0.407 | 3.72 | 83.8 |
| 2 1/3 | 13.2 | 0.407 | 2.06 | 88.5 |
| 3 1/3 | 13.4 | 0.408 | 2.04 | 89.5 |
| 4 1/3 | 13.6 | 0.416 | 2.13 | 89.2 |
| 9 | 14.2 | 0.431 | | 90.0 |
| 9 1/3 | | | 2.31 | |

CI (1) SOIL - STANDARD PROCTOR DENSITY SERIES

| | | | | |
|---|---|---|---|---|
| 0 | 15.2 | 0.590 | 2.19 | 71.4 |
| 1/3 | 15.8 | 0.592 | 1.67 | 73.8 |
| 1 1/3 | 16.2 | 0.598 | 1.30 | 75.0 |
| 2 1/3 | 16.6 | 0.605 | 1.25 | 76.0 |
| 3 1/3 | 16.7 | 0.605 | 1.04 | 76.5 |
| 4 1/3 | 17.2 | 0.621 | 1.02 | 76.7 |
| 9 | 19.1 | 0.632 | | 83.8 |
| 9 1/3 | | | 0.53 | |

CI (1) SOIL - MODIFIED PROCTOR DENSITY SERIES

| | | | | |
|---|---|---|---|---|
| 0 | 9.1 | 0.412 | 14.30 | 61.2 |
| 1/3 | 10.0 | 0.415 | 9.70 | 66.7 |
| 1 1/3 | 10.3 | 0.422 | 7.94 | 67.7 |
| 2 1/3 | 10.6 | 0.428 | 7.30 | 68.5 |
| 4 1/3 | 10.6 | 0.431 | 8.80 | 68.2 |
| 6 1/3 | 11.9 | 0.432 | 6.45 | 76.4 |
| 9 | 12.5 | 0.453 | | 76.5 |
| 9 1/3 | | | 3.93 | |

* AV. w - Average water content as determined from the changes in the weight of the specimens.
AV. e - Average void ratio as determined from the changes in the volume of the specimens.
AV. STRENGTH - Average unconfined compression strength.
AV. S - Average degree of saturation.

CI (2) SOIL - STANDARD PROCTOR DENSITY SERIES

| CYCLES | AV. w | AV. e | AV. STRENGTH | AV. S |
|---|---|---|---|---|
| 0 | 19.5 % | 0.690 % | 2.35 $kg/cm^2$ | 78.8% |
| 1/3 | 20.4 | 0.697 | 1.88 | 81.7 |
| 1 1/3 | 21.4 | 0.702 | 0.96 | 85.0 |
| 2 1/3 | 22.4 | 0.725 | 0.83 | 86.2 |
| 3 1/3 | 22.9 | 0.732 | 0.35 | 87.3 |
| 4 1/3 | 22.8 | 0.745 | 0.34 | 85.5 |
| 9 | 24.8 | 0.800 | | 86.4 |
| 9 1/3 | | | 0.42 | |

CI (2) SOIL - MODIFIED PROCTOR DENSITY SERIES

| CYCLES | AV. w | AV. e | AV. STRENGTH | AV. S |
|---|---|---|---|---|
| 0 | 12.4 | 0.461 | 10.67 | 75.0 |
| 1/3 | 14.0 | 0.482 | 9.75 | 81.0 |
| 1 1/3 | 14.2 | 0.485 | 7.65 | 81.7 |
| 2 1/3 | 14.6 | 0.490 | 6.92 | 83.0 |
| 3 1/3 | 15.0 | 0.496 | 6.54 | 84.4 |
| 4 1/3 | 14.7 | 0.494 | 5.99 | 83.0 |
| 9 | 15.3 | 0.516 | | 82.8 |
| 9 1/3 | | | 4.82 | |

CH SOIL - STANDARD PROCTOR DENSITY SERIES

| CYCLES | AV. w | AV. e | AV. STRENGTH | AV. S |
|---|---|---|---|---|
| 0 | 20.0 | 0.985 | 2.60 | 57.0 |
| 1/3 | 24.7 | 1.025 | 1.68 | 67.7 |
| 1 1/3 | 26.1 | 1.044 | 1.17 | 70.2 |
| 2 1/3 | 27.6 | 1.054 | 0.80 | 73.5 |
| 3 1/3 | 30.3 | 1.085 | 0.40 | 78.5 |
| 4 1/3 | 30.2 | 1.128 | 0.35 | 75.2 |
| 9 | 37.5 | 1.240 | | |
| 9 1/3 | | | 0.26 | 85.0 |

CH SOIL - MODIFIED PROCTOR DENSITY SERIES

| CYCLES | AV. w | AV. e | AV. STRENGTH | AV. S |
|---|---|---|---|---|
| 0 | 18.1 | 0.740 | 10.67 | 68.7 |
| 1/3 | 20.9 | 0.758 | 6.55 | 77.5 |
| 1 1/3 | 21.6 | 0.785 | 5.29 | 77.4 |
| 2 1/3 | 22.1 | 0.794 | 5.22 | 78.3 |
| 3 1/3 | 22.4 | 0.796 | 4.73 | 79.1 |
| 6 1/3 | 23.5 | 0.846 | 3.46 | 78.0 |
| 9 | 24.9 | 0.830 | | 84.3 |
| 9 1/3 | | | 3.62 | |

GRAVEL - STANDARD PROCTOR DENSITY SERIES

| CYCLES | AV. w | AV. e | AV. S |
|---|---|---|---|
| 0 | 7.4% | 0.362% | 54.8% |
| 1/3 | 11.7 | 0.393 | 79.8 |
| 1 1/3 | 12.7 | 0.401 | 85.0 |
| 2 1/3 | 13.2 | 0.412 | 86.0 |
| 3 1.3 | 13.6 | 0.431 | 84.7 |
| 4 1/3 | 14.7 | 0.446 | 88.3 |
| 5 1/3 | 14.8 | 0.450 | 88.2 |
| 6 1/3 | 15.0 | 0.455 | 88.4 |

GRAVEL - MODIFIED PROCTOR DENSITY SERIES

| | | | |
|---|---|---|---|
| 0 | 7.5 | 0.245 | 82.1 |
| 1/3 | 7.6 | 0.245 | 83.2 |
| 1 1/3 | 7.8 | 0.245 | 85.3 |
| 2 1/3 | 7.7 | 0.245 | 84.3 |
| 3 1/3 | 7.8 | 0.245 | 85.3 |
| 4 1/3 | 7.9 | 0.245 | 86.4 |
| 5 1/3 | 8.1 | 0.247 | 87.8 |
| 6 | 8.1 | 0.248 | 87.6 |